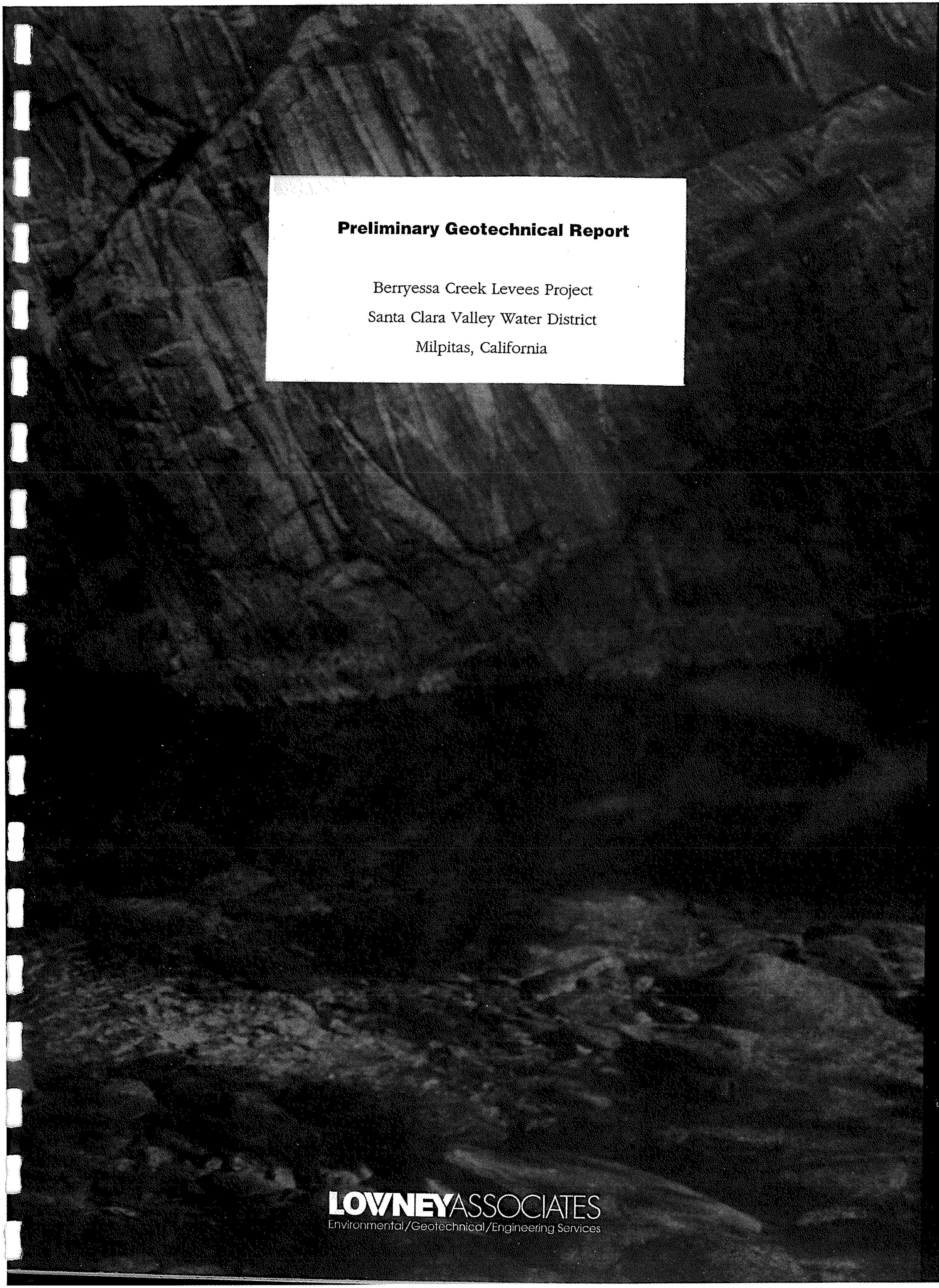

Preliminary Geotechnical Report

Berryessa Creek Levees Project
Santa Clara Valley Water District
Milpitas, California

GEOTECHNICAL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

Preliminary Geotechnical Report

Berryessa Creek Levees Project
Santa Clara Valley Water District
Milpitas, California

Report No. 1697-1 has been prepared for:

Winzler & Kelly
495 Tesconi Circle, Santa Rosa, California 95401

August 28, 2001

Robert G. Nixon, P.E.
Senior Project Engineer

Fred N. Brovold, P.E., G.E.
Senior Project Engineer

C. Barry Butler, P.E., G.E.
Principal Geotechnical Engineer
Quality Assurance Reviewer

Registered Professional Engineer
Robert G. Nixon
No. 57518
Exp. 12-31-01
Civil
State of California

Registered Professional Engineer
Frederick N. Brovold
No. GE002123
Exp. 6-30-04
Geotechnical
State of California

LOWNEYASSOCIATES
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REGISTERED PROFESSIONAL ENGINEER
ROBERT G. NIXON
No. 57518
Exp. 12-31-01
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
FREDERICK N. BROVOLD
No. GE002123
Exp. 6-30-04
GEOTECHNICAL
STATE OF CALIFORNIA

Mountain View *Oakland* *Fullerton* *San Ramon*

405 Clyde Avenue Mountain View, CA 94043-2209 Tel: 650.967.2365 Fax: 650.967.2785
P.O. Box 1388 Mountain View, CA 94042-1388 E-mail: mail@lowney.com

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

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Robert G. Nixon, P.E. Senior Project Engineer	Fred N. Brovold, P.E., G.E. Senior Project Engineer	C. Barry Butler, P.E., G.E. Principal Geotechnical Engineer Quality Assurance Reviewer

Mountain View *Oakland* *Fullerton* *San Ramon*

405 Clyde Avenue Mountain View, CA 94043-2209 Tel: 650.967.2365 Fax: 650.967.2785
P.O. Box 1388 Mountain View, CA 94042-1388 E-mail: mail@lowney.com

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PRELIMINARY GEOTECHNICAL REPORT
BERRYESSA CREEK LEVEES PROJECT
SANTA CLARA VALLEY WATER DISTRICT
MILPITAS, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation and preliminary evaluation for the engineering study phase of the Berryessa Creek Levees Project located in Milpitas, Santa Clara County, California. The alignment of the project is shown on the Vicinity Map, Figure 1. The purpose of our geotechnical investigation was to characterize the subsurface conditions in the site area, and provide geotechnical data and parameters for design of the planned levee improvements.

For our evaluation, we reviewed copies of the following materials:

- Construction Drawings, Berryessa Creek from Lower Penitencia Creek to Calaveras Boulevard, Santa Clara Valley Water District, Project 4017, March 12, 1976 (CWP-MBC-001).
- Geotechnical Report, Berryessa Creek Flood Control Improvements, U. S. Army Corps of Engineers, January 1990 (CWP-RBC-023)
- Geotechnical Engineering Analysis Report, Berryessa Creek Levee, Milpitas, California, Harza Engineering Company, Project No. M053-G, May, 1998 (CWP-RBC-021).
- Geotechnical Investigations and Recommendations for Levee Repair along Berryessa Creek, Santa Clara Valley Water District, Memorandum from Mohammad Khan, August 30, 1999 (CWP-RBC-034).

1.1 Project Description

As presently planned, the project consists of improving about 8,500 feet of the existing Berryessa Creek Levees between the confluence of Berryessa Creek and Lower Penitencia Creek, and East Calaveras Boulevard. The existing levees were constructed in 1976 and are presently about 8 to 10 feet high with a 9 to 12 foot top width, 1½:1[1] dryside slopes and 2:1 creekside slopes. A more detailed discussion of the existing levee conditions is presented in Section 2.2. The required type and extent of improvements were still being developed at the time of preparing this report; however, we understand that the improvements may require increasing the height of the existing levees 3 to 4 feet, widening the top width of the levees, steepening and reinforcing the levee slopes to remain within existing right-of-way constraints, modifying existing or constructing new hydraulic conveyance structures, and other public access and use landscaping, trail, and recreation improvements.

---

[1] All slope ratios are horizontal to vertical (H:V).

1.2 Scope of Services

Our scope of services was presented in detail in our agreement with you dated April 4, 2001. To accomplish this work, we provided the following services for the geotechnical investigation:

- Exploration of the subsurface conditions by drilling 17 borings and retrieving soil samples for observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analyses to evaluate levee embankment and foundation soil conditions, and the stability, seepage resistance, and settlement of the existing levees and improvements to the existing levees.
- Preparation of this report to summarize our findings and to present our preliminary conclusions and recommendations.

We also performed a Phase I Environmental Site Assessment for this project that is presented under separate cover.

2.0 SITE CONDITIONS

2.1 Geology

The project site is located in the Santa Clara Valley at the south end of San Francisco Bay, and within the central Coast Ranges of California (Helley, E. et al. 1989). The Santa Clara Valley is part of a long, northwest-southeast trending structural depression located between the San Andreas Fault to the west and the Hayward and Calaveras Faults to the east.

Berryessa Creek begins in the Los Buellis Hills about 7 miles upstream of the East Calaveras Boulevard Bridge. Terrain elevations near the beginning of Berryessa Creek are about 1,600 feet.

Berryessa Creek is not a natural waterway in the project area and has been artificially channeled through the area. Early topographic mapping (USGS 1898) indicates that Berryessa Creek ended in the general area about 2 miles south of East East Calaveras Boulevard and did not have a continuous creek channel extending into other more westerly creek channels or the bay. Further examination of topographic mapping indicates that Berryessa Creek appears to have been artificially channeled from this general area. Because of the conditions just discussed, the geology usually associated with stream channels such as natural streambed and alluvial deposits does not occur, except for relatively recent and shallow streambed deposits in the constructed creek channel.

Published geologic mapping indicates that the shallow surface deposits in the general project area are Holocene basin and floodplain deposits as shown on the Geology Map (Figure 2). These deposits consist of medium to dark gray, stiff to very stiff, sandy to silty clay and organic-rich clay to very fine silty clay deposits, with carbonate nodules and iron-stained mottles locally; lenses of coarser material (silt, sand, and pebbles) may also be locally present.

While not indicated on the Geology Map, recent alluvium and stream channel deposits occur within the Berryessa Creek channel.

2.2 Surface

The project site consists of the approximately 8,500-foot-long, 180- to 300-foot-wide right-of-way for Berryessa Creek and its adjacent levee system. The site is bordered by East Calaveras Boulevard to the south and Lower Penetencia Creek to the north. Within the project area, Berryessa Creek flows generally from the south to the north, approximately parallel to the Union Pacific Railway. Berryessa Creek and the railway intersect near the Abel Street Bridge.

Most of the existing project area creek alignment was constructed in 1976. Figure 3 shows a typical section for the 1976 construction. The levees were designed to be about 8 feet high with an 8-foot top width. Side slopes were 2:1 on the creekside and 1½:1 on the dryside. It appears that the fill for constructing the levees was mostly derived from excavating the creek channel section.

Berryessa Creek bottom elevations range from about 19 to 20 feet (NAVD) at the East Calaveras Boulevard Bridge to about 9 feet in the area where Berryessa Creek flows into Lower Penetencia Creek. The creek bottom slope is approximately 1.3 feet per 1,000 (0.13%) feet over the approximately 8,500 feet length of the creek in the project area.

Terrain elevations for the land adjacent to Berryessa Creek vary from about 25 to 30 feet in the vicinity of East Calaveras Boulevard to 13 to 17 feet in the general area where Berryessa Creek enters Lower Penetencia Creek. Older topographic mapping (USGS, 1961) indicates that the terrain near the confluence of Berryessa Creek with Lower Penetenicia Creek was probably between Elevation 5 to 10 feet before the area was developed, and that earthfill was probably placed to obtain the present day higher elevations of about 13 to 17 feet. Creek bottom elevations are therefore about 5 to 10 feet below the surrounding terrain near East Calaveras Boulevard and 4 to 8 feet below the surrounding terrain where Berryessa Creek joins Lower Penetencia Creek.

Figures 4 and 5 represent typical existing levee conditions. On each side of the creek the levees have a lower maintenance access road approximately 2 to 4 feet above the creek bottom elevation. Top widths of the levee range from 9 to 18 feet, and are typically 9 to 12 feet. Creekside slopes generally range from 2:1 to 2.7:1; dryside slopes range from 1½:1 to 2:1.

Beginning at the confluence of Berryessa Creek and Lower Penetencia Creek, the following sections discuss the surface conditions along the existing creek and levee alignment.

2.2.1 Confluence of Berryessa Creek and Lower Penetencia Creek to Abel Street Crossing

Photographs 1 and 2 (Figure 6) were taken near the beginning of the project, where Berryessa Creek joins Lower Penetencia Creek. Photograph 1 was taken facing easterly; Lower Penetencia Creek crosses the lower half of Photograph 1, flowing in a northerly direction and Berryessa Creek flows from east to west. Photograph 2 was taken downstream of Photograph 1, facing southerly, towards the junction of Berryessa Creek and Lower Penetencia Creek.

From the confluence of Berryessa Creek and Lower Penetencia Creek to the first bend in Berryessa Creek to the east, Berryessa Creek is paralleled on both sides by upper and lower level maintenance roads. The lower maintenance roads range in elevation from 9 to 11 feet (NAVD), and are approximately 10 to 15 feet wide. The upper maintenance roads, at the top of the levee, range in elevation from 18 to 21 feet, and are approximately 12 to 15 feet wide. The top of the existing levee is approximately 2 to 3 feet above the adjacent property elevations. Creekside embankment slopes are generally 2:1. Embankment slopes on the dryside of the levee are generally about 1½:1.

From the first bend in Berryessa Creek to the Abel Street Bridge, the westside levee embankment continues with lower and upper maintenance roads at elevations of 10 and 19 feet, respectively, 2:1 creekside slopes, 1½:1 dryside slopes, and levee top elevations 2 to 3 feet higher than adjacent properties. The eastside levee embankment slopes are approximately 2:1 from the creek bottom to the lower maintenance road. The lower maintenance road is approximately 20 to 25 feet wide, ranging in elevation from 11 to 12 feet. The upper maintenance road is 8 to 10 feet wide ranging in elevation from 19 to 20 feet. The dryside levee slopes are approximately 1½:1. The Union Pacific Railway borders the creek to the east. Railroad track elevations range from 15 to 16 feet. The upper maintenance roads are surfaced with aggregate base.

2.2.2 Abel Street Bridge

Just south of the Abel Street Bridge, Berryessa Creek makes two approximately 90-degree bends. The existing railways cross the creek at two locations between the two bends in the creek. The two bends are opposite in direction and shift the creek about 200 feet to the east. The creek channel is concrete-lined through the bend areas with side slopes of approximately 1:1 to 1½:1. Photographs 3 and 4 (Figure 7) and 5 and 6 (Figure 8) show the areas just discussed. Photographs 3 and 4 were taken facing downstream. Photographs 5 and 6 were taken looking upstream. Each photograph was taken from a position progressively more upstream than the previous photograph.

2.2.3 Abel Street Bridge to Hetch Hetchy Pipeline Crossing

Photographs 7 and 8 (Figure 9) were taken from the Milpitas Boulevard Bridge. Photograph 7 was taken looking downstream (northerly); Photograph 8 was taken looking upstream (southerly). These two photographs represent the general surface conditions along the levee from the Abel Street Bridge to the Hetch Hetchy Pipeline Crossing. The levees on each side of the creek are divided into upper and lower levels by maintenance roads. The lower maintenance roads range in width from approximately 8 feet wide, in washed out areas near the detention pond, to 25 feet in the widest areas. The upper maintenance roads range from 8 to 18 feet wide. Lower road elevations range from 12 to 18 feet. Upper maintenance road elevations range from 22 to 25 feet. Creekside embankment slopes range from 2:1 to 2½:1. Dryside slopes are approximately 1½:1.

In some areas between Milpitas Boulevard and Abel Street cracking was observed along the upper maintenance road. Cracking was primarily observed near the edges of the road. Based on probing with a 5/8-inch-diameter steel rod, the cracks range in depth from 3 to 30 inches below the ground surface and are about ¼ inch to 5 inches wide at the surface. Much of the cracking was observed on the east side of the creek, between the Hidden Lakes detention reservoir area and Milpitas Boulevard, where we understand that sprinkler systems have been installed and operated. Cracking was also observed on the westside levee, however, the cracks were generally not as wide, and not as deep as the cracks observed on the east side levee. Levee cracking is discussed further in Section 3.0; Figures 12A through 12F show the locations of observed cracking, and Figures 13 and 14 are photos of some of the cracked areas.

2.2.4 Hetch Hetchy Water Supply Pipeline Crossing:

Approximately 500 feet downstream of Hillview Drive, the Hetch Hetchy water supply pipeline crosses below Berryessa Creek. In the area of this crossing the creekside banks are lined with concrete. Ramps join the lower maintenance road to the upper level near the crossing, as shown in Photograph 9 (Figure 10). Photograph 10 shows the concrete-lined channel in the area of the pipeline crossing. Concrete slopes are approximately 2:1. The top of the levee is approximately 25 feet wide in this area. Upstream and downstream of the pipeline crossing the levee surface conditions are generally the same as the conditions found between Milpitas Boulevard and Abel Street. However, cracking along the top of the levees was generally observed less frequently than between Abel Street and Milpitas Boulevard. A few areas near the pipeline crossing showed signs of cracking, however, all cracks observed had depths less than 12 inches and widths generally less than 1 inch.

2.2.5 Hillview Drive to East Calaveras Boulevard

Between Hillview Drive and East Calaveras Boulevard creekside slopes are generally 2:1 or shallower. The lower maintenance road continues upstream of Hillview Drive, as shown in Photographs 11 and 12 (Figure 11), however, the upper maintenance road terminates at Hillview Drive. The elevation of the top of the upper creekside slope is about 30 feet, which is approximately the existing ground elevation of the adjacent properties.

2.3 Subsurface

Subsurface exploration was performed from April 16 to May 3, 2001, using conventional, truck-mounted rotary-wash drilling equipment to investigate, sample, and log the subsurface soils. Seventeen exploratory borings were drilled to depths ranging from 40 to 50 feet. The borings were permitted and backfilled in accordance with Santa Clara Valley Water District guidelines. As shown on Figures 12A to 12F, our borings and previous borings by others were spaced approximately 300 to 800 feet apart on each side of the creek. Geotechnical cross sections, summarizing pertinent geotechnical data at eight selected locations, are presented on Figures 13A to 13D. Logs of our borings and details regarding our field investigation are included in Appendix A; logs of borings by others are included in Appendix B; our laboratory test results are discussed in Appendix C.

2.3.1 Levee Fill

The existing levee fill is mostly comprised of medium stiff to stiff, high to extremely high plasticity clays and silty clays. These clays were typically moist and dark brown and gray, and mostly appear to be derived from the creek channel excavation. Table 2-1 shows the estimated thickness of levee fill at the boring locations. Levee fill laboratory test results and soil properties are discussed in Section 3.0.

2.3.2 Foundation Soils

Foundation soils immediately below the levee fill and to depths of about 8 to 20 feet below the base of the levee fill generally consist of stiff to very stiff, light brown to brown silty clays and silty clays with sand. These soils are interpreted to be alluvial fan deposits from weathering and erosion of the Coast Range mountains to the east. From a point about midway between the Hillview Avenue Bridge and the Milpitas Boulevard Bridge, the silty clay foundation soils are underlain by deposits of mostly medium dense sand, silty sand and medium stiff sandy silt. Foundation soil laboratory test results and soil properties are discussed in Section 3.0.

2.4 Ground Water

Since rotary wash drilling methods were used for this investigation, free ground water was not measured in the borings at the time of drilling. The August 30, 1999 memorandum prepared by the SCVWD indicates that two piezometers were previously installed at Borings B-4 and B-8. Ground water was measured in these piezometers at about Elevation 4 and 8 feet, respectively. Fluctuations in the level of the ground water may occur due to variations in rainfall, creek water levels, underground drainage patterns, and other factors not evident at the time our measurements were made.

Table 2-1. Estimated Fill Thickness at Boring Locations

Boring	Existing[1] Elevation (ft)	Location	Fill Depth (ft)	Clay-Silt Thickness (ft)
EB-1	24	Top	11	26
EB-3	31	Top	10	35
EB-4	27	Top	8	42
EB-7	25	Top	10	40
EB-11	22	Top	8	15
EB-12	22	Top	9	8
EB-14	19	Top	8	10
EB-15	19	Top	7	13
EB-16	18	Top	8	14
EB-17	20	Top	5	33
EB-2	20	Lower	0	40
EB-5	19	Lower	0	40
EB-6	16	Lower	0	40
EB-8	14	Lower	0	10
EB-9	14	Lower	0	11
EB-10	13	Lower	0	13
EB-13	12	Lower	0	13

[1] Taken from Site Plan topography

2.5 Levee Performance History

From discussions with District representatives (Khan, 2001), we understand that existing levee performance problems have mostly consisted of:

- the longitudinal cracking, as previously discussed,
- localized flooding in the area of the Calera Creek box culvert confluence with Berryessa Creek,
- a small area of localized creekside slope sloughing, associated with hydraulic erosion and undercutting during flooding, near the south end of the Hidden Lakes detention reservoir,
- localized erosion and slumping along the dryside of the west side levee downstream of Milpitas Boulevard associated with landscape sprinklers, and
- other areas of infrequent hydraulic erosion of the creek banks and slopes.

We further understand that the levees have not experienced other, more extensive, slope movements or failures associated with either larger flood events or earthquake shaking.

In 1995, the District placed an additional ½- to 1-foot of fill over the top area of the levee to provide additional freeboard to the levee. In 1999, the District filled existing longitudinal cracking with a controlled sand-cement low strength fill.

3.0 SOIL PROPERTIES FOR EVALUATION AND DESIGN

The engineering properties of the levee fill and foundation soils were evaluated by visually examining and classifying soil samples and performing a laboratory testing program. Soil properties for evaluation and design are summarized and discussed below.

3.1 Levee Fill

As discussed in the preceding section, the existing levee fill is mostly comprised of medium stiff to stiff, high to extremely high plasticity clays and silty clays. As discussed in Section 2.3.1, most of the levee fill appears to be from excavations for the creek channel and consist of high plasticity clays derived from basin deposits.

3.1.1 Atterberg Limits

Table 3-1 summarizes plasticity test results for the levee fill. Liquid Limits for the levee fill ranged from 36 to 124 and Plastic Limits ranged from 17 to 29. Plasticity Indexes ranged from 19 to 103 and were mostly greater than 30, indicating high to extremely high plasticity and expansion potential.

3.1.2 Moisture and Density

Table 3-2 summarizes moisture content and density test results for the levee fill. Moisture contents in the levee fill mostly ranged from about 15 to 27 percent and averaged about 21 percent. Dry densities mostly ranged from about 94 to 114 pounds per cubic foot (pcf) and averaged about 104 pcf; moist or wet densities mostly ranged from 120 to 132 pcf.

3.1.3 Compaction

Five compaction tests (ASTM D1557) were performed on samples of the levee fill. Maximum dry densities ranged from 110 to 119 pcf and optimum moisture contents ranged from 15 to 19 percent.

3.1.4 Shear Strength

Shear strength of the levee fill was evaluated using Torvane and Pocket Penetrometer testing devices and from laboratory testing. Laboratory shear strength included:

- triaxial unconsolidated undrained tests to estimate the undrained shear strength for the "end of construction" condition or other short term loading conditions, and

- triaxial consolidated-undrained shear tests with pore pressure measurements to estimate both undrained and drained shear strengths.

Table 3-3 summarizes triaxial shear test results for the levee fill and foundation soils; undrained shear strengths estimated from approximate Torvane and Pocket Penetrometer testing are summarized on Table 3-2. Undrained shear strengths from the triaxial UU tests ranged from 600 to 1,100 psf. Torvane and Pocket penetrometer undrained shear strengths ranged from about 600 to 2,300 psf and averaged about 1,100 to 1,300 psf.

Effective stress cohesion and friction angle strength parameters were:

Effective cohesion (c'):	0 psf
Effective friction angles (Ø'):	37° – 45°

3.1.5 Compressibility

Four laboratory consolidation tests were performed to evaluate the compressibility of the levee fill. Table 3-4 summarizes the consolidation test data for the levee fill and foundation soils. Compression ratios for the levee fill ranged from 0.003 to 0.011 for stresses less than maximum past stress conditions and 0.14 to 0.16 for stresses greater than the maximum past stress.

3.1.6 Hydraulic Conductivity

Five levee fill samples were tested for hydraulic conductivity. Table 3-5 summarizes the hydraulic conductivity test data for the levee fill and foundation soils. Hydraulic conductivity test results on samples of the levee fill mostly ranged from 1×10^{-6} to 1×10^{-8} cm/sec indicating very low hydraulic conductivity. One sample exhibited a hydraulic conductivity of 3×10^{-4} cm/sec, however, this sample is not considered representative of most of the levee fill encountered in the borings.

3.1.7 R-value

Three levee fill samples were tested to determine their R-values for pavement thickness design. Two of the samples exhibited an R-value less than 5 and one sample exhibited an R-value of 6. These are very low R-values indicating that the levee fill is a poor subgrade material that will generally require thicker aggregate surfacing or pavement structural sections.

3.1.8 Geogrid Interaction Coefficients

Three bulk samples of the levee fill soils, representing the range of plasticity exhibited by the levee fill, are being tested with samples of Tensar geogrid materials to evaluate interaction coefficients. This testing was still in progress at the time of preparing this report and will be provided as a supplement to this report when it is completed.

3.2 Foundation Soils

As discussed in the preceding section, foundation soils immediately below the levee fill and to depths of about 8 to 20 feet below the base of the levee fill mostly consist of stiff to very stiff, light brown to brown silty clays and silty clays with sand. These soils are interpreted to be alluvial fan deposits from weathering and erosion of the Coast Range mountains to the east.

From a point about midway between the Hillview Avenue Bridge and the Milpitas Boulevard Bridge, the silty clay foundation soils are underlain by deposits of mostly medium dense sand, silty sand and medium stiff sandy silt.

3.2.1 Atterberg Limits

Liquid Limits for the fine-grained foundation soils ranged from 23 to 82 and Plastic Limits ranged from 15 to 22. Plasticity Indexes ranged from 2 to 60 and were mostly between 10 and 30 indicating moderate to high plasticity and expansion potential.

3.2.2 Moisture and Density

Table 3-6 summarizes moisture content and density test results for the foundation soils. Moisture contents of the fine-grained foundation soils mostly ranged from about 17 to 28 percent and averaged about 23 percent. Dry densities mostly ranged from about 96 to 113 pounds per cubic foot (pcf) and averaged about 104 pcf; moist or wet densities mostly ranged from 121 to 133 pcf and averaged 127 pcf.

3.2.3 Compaction

Compaction tests were not performed on any of the foundation soils because they were not expected to be excavated and used for fill.

3.2.4 Shear Strength

Shear strength of the fine-grained foundation soils was evaluated using Torvane and Pocket Penetrometer testing devices and from laboratory testing. Laboratory shear strength testing included five consolidated undrained shear tests with pore pressure measurements to estimate both undrained and drained shear strengths.

Undrained shear strengths from the triaxial CU tests ranged from 800 to 1,500 psf. Torvane and Pocket penetrometer undrained shear strengths ranged from about 1,100 to 5,000 psf and averaged about 1,700 to 2,200 psf.

Effective stress cohesion and friction angle strength parameters were:

Effective cohesion (c'): 0 psf

Effective friction angles (ø'): 32° – 39°

3.2.5 Compressibility

Three laboratory consolidation tests were performed to evaluate the compressibility of the fine-grained foundation soils. Compression ratios for samples of these foundation soils ranged from 0.004 to 0.007 for stresses less than maximum past stress conditions and 0.11 to 0.13 for stresses greater than the maximum past stress.

The laboratory test results indicate that the foundation soils are slightly less compressible than the levee fill.

3.2.6 Hydraulic Conductivity

Five foundation soil samples were tested for hydraulic conductivity. Hydraulic conductivity test results on samples of the foundation soil mostly ranged from 4 x 10^{-4} to 2 x 10^{-8} cm/sec indicating low to very low hydraulic conductivity.

3.2.7 R-value

R-value tests were not performed on any of the foundation soils because they were not expected to be used for access road subgrades.

Table 3-1. Levee Fill Plasticity Summary

Boring	Sample	Depth (ft.)	In-place Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Soil Classification Group	Description
EB-1	4A	8.5	36	83	29	54	CH	Black clay
EB-1	6A	14.5	27-31	82	22	60	CH	Light brown clay
EB-2	1A	2.0	19-23	36	17	19	CL	Light brown silty clay
EB-3	2A	4.5	29-31	71	26	45	CH	Black clay
EB-3	4A	10.5	24-28	49	19	30	CL	Light gray clay
EB-7	3A	7.5	31	75	22	53	CH	Dark brown silty clay
EB-14	1A	2.0	17	44	17	27	CL	Brown silty clay
EB-17	3A	5.8	24	39	20	19	CL	Brown clay with sand
					Min=	19		
					Average=	38		
					Max=	60		

Table 3-2. Summary of Levee Fill Moisture, Density, Torvane and Pocket Penetrometer Data

							Shear Strength	
	Fill Depth (feet)	**Sample Number**	**Depth (feet)**	**Wet density (pcf)**	**Moisture (%)**	**Dry density (pcf)**	**Torvane (ksf)**	**Pocket Pen. (ksf)**
EB-1	11	1A	2.0	125	23	101	2.0	2.5
EB-1	11	3A	6.5		19			4.5
EB-3	10	1A	1.5	123	22	101	3.0	2.5
EB-3	10	3A	8.0	121	19	102	2.2	2.8
EB-4	8	1A	2.0	135	10	122		4.5
EB-4	8	2A	4.5	120	28	94	3.6	3.3
EB-7	10	1B	2.0	123	22	100	2.0	2.8
EB-7	10	3A	7.5	108	31	83	2.0	2.4
EB-11	8	1A	2.0	130	17	111	1.6	3.7
EB-11	8	3A	7.0	122	24	98		2.8
EB-12	9	1B	2.0	131	12	117	3.0	3.5
EB-12	9	2B	4.5	129	30	99		2.0
EB-14	8	1A	2.0	132	17	113	2.0	3.9
EB-14	8	2A	4.5	130	25	104		2.3
EB-14	8	2B	6.5	126	25	101		3.1
EB-15	7	1A	2.0	127	16	110	3.0	4.5
EB-15	7	3A	7.0	117	29	90		2.5
EB-16	8	1A	2.0	124	23	101	1.6	2.8
EB-16	8	2A	4.5		26		2.0	1.5
EB-17	5	1A	2.0	128	13	113		3.5
EB-17	5	2B	4.5	134	19	113		3.1
		Min=		108	10	83	1.6	1.5
		Average=		126	21	104	2.4	3.1
		Max=		135	31	122	3.6	4.5
		Std Dev=		6	6	10	0.6	0.8

Table 3-3. Summary of Soil Shear Strengths

Boring	Sample	From (ft)	To (ft)	Dry Density (psf)	Moisture (%)	Approx. Wet Density (pcf)	Approx. Vertical Effective Shear Strength (psf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective ϕ (degrees)
Levee Fill										
EB-3	2A	3.5	6	85-89	29-31	112	450	600	0	45
EB-7	2A	3	6	90-104	12-30	117	450	600	0	44
EB-16	3A	5	8	96-100	23-25	122	720	1100	0	
EB-1	2A	2.5	6	199	20			580		
EB-4	3A	5	8	99	25			1600		
EB-11	5A	3	6	100	24			1000		
EB-15	2A	3.5	7	92	27			640		
Foundation Clays										
EB-1	6A	13	16	91-94	27-31	119	1700	1000	0	32
EB-3	6A	15	18	102-105	21-23	126	2100	1500	0	37
EB-7	7A	16	19	107-112	18-20	130	1700	900	0	39
EB-9	5A	12	15	103-105	25-26	129	1000	1200	0	37
EB-14	5A	12.5	16	106	21			1300		
EB-15	4A	11	14	100-109	19-22	126	1570	780	0	32
EB-16	5A	10	13	100	23			620		
EB-17	5A	10	13	94	30			1360		
EB-17	6A	13	16	87	32			650		
Foundation Sands										
EB-12	10A	31	34	88-93	29-32	116	2400			33

Table 3-4. Consolidation Test Summary

Boring	Sample	From (feet)	To (feet)	Dry Density (pcf)	Moisture (%)	CR		Strain @		Soil Description
						Initial	Virgin	1,000 psf	5,000 psf	
Levee Fill										
EB-1	4A	7	10	83.8	36.3	0.011	0.16	0.8	5.7	Black clay
EB-3	4A	9	12	91.9	24.3	0.003	0.15	0.3	4.8	Light gray clay
EB-17	3A	5.6	5.8	105.7	21.2	0.006	0.14	0.7	2.9	Dark brown clay w/ sand
Foundation Soils										
EB-2	2A	3	6	106.1	22.1	0.004	0.13	0.6	3.0	Brown clayey sand
EB-7	7A	16	19	109.7	18.3	0.007	0.11	0.7	3.5	Brown clayey sand
EB-9	5A	12	15	103.4	24.6	0.005	0.12	1.1	3.2	Brown sandy silt

Table 3-5. Hydraulic Conductivity Summary

Boring	Sample	From (ft)	To (ft)	Dry Density (pcf)	Moisture Content (%)	Effective Stress (psf)	Hydraulic Conductivity (cm/sec)	Description
Levee Fill								
EB-1	4A	7	10	77	38	990	2.0×10^{-7}	Black clay
						2000	3.0×10^{-8}	
EB-3	4A	9	12	85	28	990	4.0×10^{-4}	Light gray clay
						2000	3.0×10^{-4}	
EB-11	2A	3	6	96	27	580	2.0×10^{-8}	Brown clay
						1150	1.0×10^{-8}	
EB-15	5A	3.5	6.5	96	27	500	2.0×10^{-8}	Brown clay w/sand
						1000	2.0×10^{-8}	
Foundation Soils								
EB-2	2A	3	6	101	23	990	3.0×10^{-7}	Brown sandy clay
						2000	2.0×10^{-7}	
EB-3	6A	15	18	114	15	1500	7.0×10^{-5}	Light brown clayey sand
						3000	6.0×10^{-5}	
EB-9	5A	12	15	100	25	800	7.0×10^{-7}	Brown clayey silt w/sand
						1500	5.0×10^{-7}	
EB-17	3A	5	8	97	24	800	9.0×10^{-8}	Dark brown clay w/sand
						2000	2.0×10^{-8}	

Table 3-6. Foundation Clay Summary

Boring	Sample	Depth (feet)	Wet Density (pfc)	Moisture (%)	Dry Density (pcf)	Shear Strength	
						Torvane (ksf)	Pocket Pen. (ksf)
EB-1	5A	12	119	29	92	1.6	2.2
EB-1	7A	19.5	124	25	99	1.2	1.3
EB-1	8A	24.5	120	32	91	1.2	1
EB-2	1A	1	130	19	109		5
EB-2	3A	6	128	21	106		2
EB-2	4A	8.5	129	22	106	1.6	1
EB-2	7A	19.5	132	19	111	1.2	0.5
EB-2	11A	39.5	132	16	114		4.5
EB-3	5A	14.5	124	23	101	1.6	1.5
EB-3	11A	39.5	135	16	116		2.5
EB-4	4A	9.5	123	21	102		2
EB-4	8A	24	137	18	116	1.8	2.1
EB-4	11A	39.5	131	18	111	1	1.1
EB-4	13A	49.5	132	20	110	2.2	2.8
EB-5	1B	1.5	123	17	105		3.5
EB-5	2B	4	123	23	100		2.5
EB-5	3B	6.5	131	19	110		4
EB-5	6B	19.5	133	16	115	2.6	3.8
EB-5	7B	24.5	130	19	109	1	1.1
EB-5	10A	39	132	17	113		1.5
EB-6	1B	1.5	119	20	99	2	2
EB-6	2B	4	120	24	97	1.6	1.5
EB-6	3B	6.5	127	20	106	2.2	3.5
EB-6	5A	14	134	22	110		3.1
EB-6	7A	23.5	131	20	109		3.2
EB-6	9A	33.5	135	16	116	2.6	3.5
EB-7	5A	12	129	22	106	2.6	3
EB-7	8A	21	132	17	113	2.6	3.9
EB-7	10A	31	133	17	114		4.5
EB-7	12A	41	132	17	113	2.2	2.5
EB-8	2B	4	118	31	90	1.2	1.3
EB-8	3B	6.5	127	21	105	1.6	1.5
EB-9	2A	4	120	30	92	1.8	1.8
EB-10	1B	2	118	27	93	1.6	1.6
EB-10	2A	4.5	118	33	89	1.6	1.6
EB-10	4A	9.5	125	24	101	1	1.4
EB-10	10A	36	131	20	109	1.6	2

continued

Table 3-6. Foundation Clay Summary
(continued)

Boring	Sample	Depth (feet)	Wet Density (pfc)	Moisture (%)	Dry Density (pcf)	Shear Strength	
						Torvane (ksf)	Pocket Pen. (ksf)
EB-11	6A	14.5	119	32	90	1.2	1.3
EB-11	7A	19.5	129	23	105	1.6	1.2
EB-11	13A	46	132	21	109	1.6	1
EB-12	13A	46	136	25	109		1.4
EB-13	1A	2	119	29	92	2.6	2.1
EB-13	2A	4.5	113	39	81	1.2	1.2
EB-13	4A	9.5	129	24	104	1.8	2.6
EB-13	12A	39.5	132	20	110	1.0	1.3
EB-14	1A	2	132	17	113		3.9
EB-14	2B	6.5	126	25	101	3.0	3.1
EB-14	3A	12	115	35	85	1.8	2.4
EB-14	4A	15	126	30	97	1.0	1.7
EB-14	11A	48.5	130	19	109	1.6	1.5
EB-15	5A	14.5	132	20	110	1.8	2.2
EB-15	11A	41	128	21	106	1.2	1.4
EB-16	6A	13.5	133	17	114	1.8	2.1
EB-16	13A	44.5	124	22	102		0.1
EB-17	4A	9.5	123	22	101	1.8	2.8
EB-17	14A	49.5	124	29	96	1.6	1.7
		Average	127.2	22.5	104.1	1.7	2.2
		Max	136.9	39.0	116.0	3.0	5.0
		Min	112.6	16.0	81.0	1.0	0.1
		Std Dev	6.0	5.4	8.7	0.5	1.1

4.0 GEOLOGIC HAZARDS

Geologic hazards include earthquake faulting and ground shaking, ground liquefaction, differential compaction, lateral spreading, and flooding. The following sections discuss these geologic hazards.

4.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented on Figure 15. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). As shown on Figure 15, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

4.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to most projects in the San Francisco Bay Area. The "Seismicity" section that follows (Section 5.0) summarizes potential levels of ground shaking at the site.

4.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands.

We performed a liquefaction analysis on the sands identified during this study using the method proposed by Seed and Idriss (1982) and the proceedings of the National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils (1997). This analysis was performed using a mean peak ground acceleration value of 0.45g and our experience with similar projects. The results of our analysis indicate that there is moderate to high potential for some of these sands to liquefy during a major earthquake on a nearby fault. Since the boring data indicate that these potentially liquefiable sands are discontinuous, only localized areas of liquefaction are anticipated.

Based on a recent study by Youd and Garris (1995), which expanded on the work of Ishihara (1985) to include data from 308 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration, there is a sufficient clay cap over these potentially liquefiable sands to reduce the potential for surface expression in the form of sand boils or small fissures. Therefore, if localized liquefaction were to occur, we estimate that it would be manifested as uneven ground settlements ranging from less than an inch to about three inches.

4.4 Differential Compaction

If near-surface, unsaturated soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. Because the near-surface soils encountered at the site are generally stiff clays and do not appear to change in thickness or consistency abruptly over short distances, we judge the probability of significant differential compaction at the site to be low.

4.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

We evaluated the potential for lateral spreading by estimating a lower range residual shear strength for the sands that occur at depths of 12 to 32 feet below the ground surface. These analyses are discussed in more detail in Section 6.0 and show that factors of safety are greater than 1.2 for this condition. For this reason, we judge that the probability of lateral spreading occurring at the site during a seismic event is low.

4.6 Flooding

Flooding may be caused by intensive rainfall, tsunami or seiche, or dam or levee breaks. The terms tsunami and seiche describe ocean tidal waves or similar waves in closed bodies of water.

Intensive Rainfall: The potential for flooding due to intensive rainfall is being evaluated by others as part of the engineering study phase of this project.

Tsunami or Seiche: The site is several miles inland from the San Francisco Bay shoreline. Hence, the potential for inundation due to tsunami and/or seiche is considered remote.

Dam Break: As discussed in the March 1982 Santa Clara County General Plan, there are twelve major dams in Santa Clara County. None of these dams were expected to inundate the site area if a failure occurred.

5.0 SEISMICITY

5.1 Regional Active Faults

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The San Andreas Fault, which generated the great San Francisco earthquake of 1906, passes about 16 miles west of the site. Three other major active faults to the east of the project area are the Hayward Fault (Southeast Extension), located about 1½ miles east, the Hayward Fault, located about 4 miles northeast, and the Calaveras Fault, located about 5½ miles southeast.

5.2 Maximum Estimated Ground Shaking

We performed a Deterministic Seismic Hazard Analysis (DSHA) for the project location. This analysis utilizes the maximum Moment Magnitude (M_w) for the controlling faults, published attenuation curves, the shortest distance to the fault, and site-specific response characteristics. Based on the attenuation methods of

Campbell and Bozorgnia (1994), the mean peak ground acceleration (PGA) expected for this site is 0.45g from a magnitude 6.4 M_w event on the Hayward (Southeast Extension) Fault. If a 6.8 M_w event were to occur on the Calaveras Fault, the expected PGA is 0.40g.

5.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (1999), referred to as WG99, determined that there is a 70 percent chance (±10%) of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2000 and 2030. This result is the most important outcome of WG99's work, because any major earthquake can cause damage throughout the region. This was dramatically demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can inflict damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

5.4 UBC Site Coefficient

Based on our borings and alluvium thickness maps of Santa Clara County (Rogers and Williams 1974), the site is underlain by stiff soils extending to depths of about 300 feet. The California Division of Mines and Geology (CDMG) has recently issued maps locating "Active Fault Near-Source Zones" to be used with the 1997 Uniform Building Code ("Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," CDMG/ICBO February 1998). Faults are classified as either "A," "B," or "C" as shown in Table 5-1. Only faults classified as "A" or "B" are mapped since faults classified as "C" do not increase the near-source factor.

Table 5-1. Seismic Source Definitions

Seismic Source Type	Seismic Source Description	Seismic Source Definition*	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/yr)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C.	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$	$SR \leq 2$

*Note: Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining seismic source type.

Table 5-2 lists Type A and Type B faults within 25 kilometers of the site:

Table 5-2. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (kilometers)
**Hayward (Southeast Extension)	B	2.2-3.0
*Hayward (Total Length)	A	6.3-6.8
Calaveras (South)	B	8.9-11.0
Monte Vista - Shannon	B	20.4-20.8
San Andreas (1906)	A	25.9-26.5

*Nearest Type A fault
**Nearest Type B fault

Based on this information, the site may be characterized for design based on Chapter 16 of the 1997 UBC using the information in Table 5-3.

Table 5-3. 1997 UBC Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_D
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Hayward (SE Extension)
Seismic Source Type (Table 16-U)	B
Distance to Seismic Source (kilometers)	2.2-3.0
*Near Source Factor N_a (Table 16-S)	1.28
Near Source Factor N_v (Table 16-T)	1.57
Seismic Coefficient C_a (Table 16-Q)	0.56
Seismic Coefficient C_v (Table 16-R)	1.01

*Note: For Seismic Zone 4, the near-source factor Na used to determine Ca need not exceed 1.1 for structures complying with all the conditions within UBC Section 1629.4.2.

6.0 PRELIMINARY GEOTECHNICAL EVALUATION

The following evaluation is called preliminary because required design water surface elevations and channel flow velocities were still being developed at the time of preparing this report and because other work associated with the engineering study phase of the project will develop and evaluate levee flood protection alternatives and determine a preferred alternative for further analysis and design.

Objectives of the preliminary geotechnical evaluation were to:

- assess the materials (soils), stability, settlement, and erosion protection of the existing levees

- evaluate the existing longitudinal cracking that occurs at some locations along the project levee alignment

- provide preliminary recommendations for maximum levee slopes and minimum levee crown width using the existing levee materials.

6.1 Federal Emergency Management Agency Levee Geotechnical Design Criteria

Federal Emergency Management Agency (FEMA) levee geotechnical design criteria are provided in Appendix D and summarized and discussed below. Some parts of the FEMA design criteria cross reference U.S. Army Corps of Engineers (COE) criteria and methods also included in Appendix D. Also included in Appendix D are the geotechnical design criteria prepared by the Coyote Watershed Project for the Lower Silver Creek project (CWP, 2001) which generally conform with FEMA design criteria.

Freeboard – Freeboard refers to the vertical distance between the top of the levee and the water surface elevation for the design basis flood. FEMA generally requires a minimum freeboard of 3 feet. An additional 1 foot above the minimum is required within 100 feet on either side of structures, such as bridges, on the creekside of the levee.

In the past, the COE had similar freeboard requirements. More recently, the COE has changed to a risk-based evaluation of water surface levels coupled with a deterministic levee design for the geotechnical conditions.

Embankment Erosion Protection – FEMA requires that engineering analyses be provided that demonstrate that no appreciable erosion of the levee embankment can be expected during the design basis flood as a result of currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability.

Embankment and Foundation Stability – FEMA requires engineering analyses that evaluate levee embankment stability. These analyses are generally performed to conform with COE requirements for End-of-Construction (Case I), Sudden Drawdown (Case II), Steady Seepage at Base Flood Level (Case III), and Case I with Earthquake (Case IV) (COE, 2001).

Minimum required factors of safety for these analysis conditions are shown in Table 6-1. The end of construction case would be applicable to both creekside and dryside slopes for newly constructed levees. The sudden drawdown case applies to the creekside slope and represents the condition where the design flood stage saturates part or all of the levee depending on the duration of flooding and hydraulic conductivity properties of the levee and foundation soils. The steady seepage from full design flood stage represents a condition where the duration of the full flood design water elevation is long enough that the embankment soils become fully saturated and a condition of steady seepage occurs through the embankment. This condition, if the flood duration is long enough, may be critical for the dryside slope. However, the duration of the design level flood may not be long enough for this condition to develop. In this case, seepage analyses may be used to estimate the extent of saturation advance through the embankment and the stability of the dryside slope evaluated for the intermediate condition.

Earthquake loadings are not normally required to evaluate the stability of levees for the long-term (steady seepage) case at full flood because of the low probability of an earthquake coinciding with flooding conditions.

Settlement – Engineering analyses are required that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that the minimum required freeboard will be maintained.

6.2 Levee Fill

Soil types, classifications, and properties of the existing levee fill have been discussed in Sections 2.0 and 3.0. The levee fill, as sampled in the borings, is generally a relatively strong, moderately compressible, very low hydraulic conductivity soil. With the exception of high plasticity, discussed in more detail below, characteristics and properties of the levee fill are mostly favorable for levee stability and performance.

The levee fill mostly consists of black to dark brown medium stiff to stiff, very high plasticity silty clay. Dry densities of samples of the levee fill ranged from about 83 to 122 pcf and averaged about 104 pcf. The plus or minus one standard deviation dry density range of the levee fill is 94 pcf to 114 pcf. Compaction test (ASTM D1557) maximum dry densities of samples of the levee fill ranged from about 110 to 119 pcf. Relative compaction of the levee fill based on dry densities of 94, 104, 114 pcf are approximately 79 to 85 percent, 87 to 95 percent, and 96 to 103 percent, respectively.

The least favorable characteristic or property of the levee fill is its high to very high plasticity. The high plasticity of the levee fill appears to be one of the major factors resulting in the observed shallow shrinkage cracking. Based on probing the cracks with a steel rod, the maximum depths of the cracking are about 2½ feet. In most areas, the shrinkage cracking tends to be longitudinal along the levee, as shown in Figures 13 and 14. For our analysis of the existing levees, we assumed that the upper 2½ feet of levee fill would have practically no strength contribution and that the levee slopes could be periodically shaped and regraded.

The following sections discuss seepage resistance, slopes and stability, and stability of the levee embankments. Our seepage resistance evaluation is discussed first because the results of the seepage analysis are used for evaluating and discussing levee slopes and stability.

6.3 Seepage Resistance

Figure 15 shows the typical levee section that we used for evaluating seepage resistance and stability of the existing levee. As previously discussed, creekside slopes typically vary from a little steeper to a little flatter than 2:1 and dryside slopes typically vary from a little steeper to a little flatter than 1½:1.

The levee fill was modeled as homogeneous clay with both a vertical and horizontal hydraulic conductivity of 1 x 10^{-6} cm/sec (2.8 x 10^{-3} feet/day). A vertical to horizontal ratio of the hydraulic conductivities was not used due to the observed homogeneous characteristic of the levee fill samples. Most of the laboratory test results on samples of the levee fill ranged from 2 x 10^{-7} to 1 x 10^{-8} cm/sec. One sample that was tested, boring EB-3 @ 9 to 12 feet, was not considered to be representative of most of the levee fill and exhibited a hydraulic conductivity of 3 to 4 x 10^{-4} cm/sec. We chose a higher hydraulic conductivity of 1 x 10^{-6} cm/sec for the levee fill as a conservative estimate of the levee fill hydraulic conductivity.

Laboratory test results on samples of the clayey foundation soils ranged from 7 x 10^{-5} cm/sec to 2 x 10^{-7} cm/sec. We selected a higher value of 1 x 10^{-4} cm/sec to model the foundation soils.

The computer program SEEP/W was used for the seepage analyses; SEEP/W performs seepage analyses by the finite element method. Results of the seepage analyses are summarized in Appendix E. For the seepage analyses, the upper 2½-foot zone of cracked and weakened soil was assigned a higher hydraulic conductivity of 1 x 10^{-4} cm/sec and the rest of the levee fill was assigned a hydraulic conductivity of 1 x 10^{-6} cm/sec. The figures in Appendix E show the advance of the wetting front or zone of saturation at different times (days) after flooding. Figures E-1 and E-2 are for the existing conditions with the creek water surface near the top of the levee (Figure E-1) and 3 feet below the top of the levee (Figure E-2). Figures E-3 and E-4 are for modified levee section with a 14-foot top width and 50°, from horizontal, levee slopes. The ground water level before flooding was considered to be at the creek channel bottom elevation.

On Figure E-1, the seepage modeling indicates that it would take much longer than 40 days for a fully-saturated steady seepage condition to develop with a creekside water level about 6 inches below the top of the existing levee, whereas on Figure E-3, with a cracked and weakened upper zone, the seepage modeling indicates that the wetting front would advance through the upper zone in about 20 days. For both of the cases shown on Figures E-1 through E-4, the bulk of the levee fill does not become fully saturated during the expected flood duration of a few days to one week and the dryside slope would therefore not be subject to a fully developed steady seepage condition. Results of these analyses were subsequently used to estimate the piezometric surface in the levee cross section for stability analyses. These results are discussed in a following section.

6.4 Levee Embankment Slopes and Stability

Stability refers to the capacity of the levee and foundation soils to resist shear stresses resulting from gravity forces acting on the sloping levee geometry. Factors of safety are usually expressed in terms of the ratio of resisting capacity or strength divided by driving forces and moments.

We used the computer program SLOPE/W for our slope stability analysis. SLOPE/W is a limit equilibrium program that computes the factors of safety for a wide range of circular, irregular and block failure surfaces using various analysis methods. For our analyses, we used the Modified Bishop's and Spencer's Methods run simultaneously with the Method of Slices and Janbu Methods to determine sliding surfaces with a minimum factor of safety. SLOPE/W uses an iterative search routine to locate the minimum factor-of-safety and sliding surface. The search routine is modified for interactive user input, allowing hundreds of potential failure surfaces to be analyzed efficiently.

Table 6-2 summarizes the soil properties used for the stability analyses. As previously discussed, to account for the observed longitudinal cracking along parts of the existing levee, the stability analyses considered the top 2½ feet of the levee to have no strength.

Calculated factors of safety for the existing levee slopes and analysis conditions are provided in Table 6-3. All of the estimated factors of safety exceed the minimum required factors of safety. Figures showing the stability analysis conditions and sliding surfaces with the lowest factors of safety are provided in Appendix E.

Case I – End of Construction (Short and Long Term) – Per COE guidelines, the levee fill and clay foundation soils are classified as low permeability soils that should be analyzed using undrained strengths and total stresses for the end-of-construction case. Since the levees have been constructed for many years, we analyzed the end of construction case for two conditions called short and long term. For the short-term condition, we used undrained strengths for both the levee fill and foundation soils. For the long-term condition, we used undrained strengths for the levee fill and drained strengths for the foundation soils.

For the short-term condition, the stability analyses indicate factors of safety of 2.2 for the creekside slopes and 2.4 for the dryside slopes. For the long-term condition, the stability analyses indicate factors of safety of 2.1 for the creekside slopes and 2.3 for the dryside slopes.

Case II - Sudden Drawdown – The COE currently recommends analyzing sudden drawdown stability using before drawdown pore pressures and either drained strengths and effective stresses or undrained strengths with pore pressures set to zero depending on whether the drained or undrained strengths are less. The previous COE method used a bilinear shear strength envelope that is a combination of drained strength (slow or S tests) at stress levels below the normal stress intersection of the two strength envelopes and consolidated-undrained strengths (rapid or R tests) at stress levels above the envelope intersection normal stress. Since the previous COE method requires less computational effort and has been shown to be more conservative than the current COE method, we calculated the sudden drawdown stability using the previous COE method and found that the factor of safety (1.8) was greater than required, making is unnecessary to use the current COE method (Duncan, et al, 1990).

Case III – Steady Seepage from Full Flood Stage – This condition is usually more important when the duration of flooding or water retention is long enough for seepage to advance through the levee embankment and emerge from the dryside slope of the levee. The COE advises using drained strengths and effective stresses for the steady state seepage case for saturated soils. Because the duration of flooding for the Berryessa Creek levees is expected to be a few days to a week and the embankment soils have a very low hydraulic conductivity, seepage analyses indicate that the seepage wetting front only advances a few feet into the creekside embankment slope, and consequently, only a small portion of the creekside embankment soils may potentially become saturated. The dryside slopes are therefore not affected by seepage from the expected short flood durations and the factor of safety (2.3) is similar to that for long-term stability without flooding.

Case IV – Earthquake – Recent COE guidance advises "earthquake loadings are not normally considered in analyzing the stability of levees at full flood stage because of the low probability of an earthquake coinciding with periods of high water." We analyzed the stability of the levees with earthquake forces for short term and long term conditions an unflooded condition. We used undrained strengths and total stresses for the low permeability levee fill and foundation soils and drained strengths and effective stresses for the sand foundation soils. We also used a psuedo-static earthquake coefficient of 0.2g in accordance with recent District guidance (CWP, 2001).

The stability analyses indicate short and long term condition factors of safety of 1.5 and 1.4 for the creekside slopes and 1.6 and 1.5 for the dryside slopes. These factors of safety exceed the required minimum factor of safety of 1.0.

Because our liquefaction analyses indicate that some of the foundation soils, consisting of mostly medium dense sands and silty sands, are potentially liquefiable under strong earthquake-induced ground shaking, we evaluated the stability of the levee and foundation cross section using post liquefaction residual strengths for the foundation sands. Our evaluation of liquefaction potential has been previously discussed in the "Geologic Hazards" section.

For the post-earthquake liquefaction stability analysis, we analyzed stability using post-liquefaction residual strengths, without earthquake forces, and calculated a factor of safety of 2.0. We also assigned a post-liquefaction residual strength of 240 psf to the liquefied foundation sands (Seed and Harder, 1990) and applied a horizontal pseudo-static earthquake coefficient of 0.2g. This analysis indicated a factor of safety of 0.8.

Based on these analyses, our preliminary conclusion is that, if the foundation sands liquefy, there is some risk of permanent levee and surrounding ground horizontal deformation ranging from a few inches to about one foot; and, vertical deformation, that is, settlement, ranging from less than an inch to 3 inches. We recommend that more detailed stability and deformation analyses be performed during final engineering design to further evaluate our preliminary conclusions.

6.5 Previous Stability Analyses

The stability of the Berryessa Creek levees was previously evaluated by others (Harza, 1998). Where the cases and conditions were similar, Table 6-3 also provides factors of safety for the previous analysis results in brackets. The factors of safety are generally similar for the end-of-construction condition but differ for the sudden drawdown and steady seepage cases. For the sudden drawdown case, the differences are attributed to differing strength properties used for the analyses. For the steady seepage condition, the differences appear to result from different seepage model analysis results with regard to the time required for a steady seepage condition to develop.

The previous stability analyses also indicated some other factors of safety that were much greater (not shown in Table 6-2) than resulting from our analyses. Most of the higher factors of safety generally appear to result from analyzing some slopes that were flatter than the typical steeper slope geometry that we selected for our analyses or from analyzing both the creekside and dryside slopes for the different cases rather than the most critical slope for a particular case. The previous analyses also used different strengths for some of the soils and did not appear to include an upper shallow zone of soils weakened by cracking.

6.6 Observed Levee Cracking

With the exceptions discussed above, the stability analysis results indicate relatively high factors of safety for the existing levee condition and support our conclusion that the existing longitudinal cracking most likely results from shrinkage of the high-plasticity levee fill, shallow slope creep of an upper shallow zone of cracked and weakened soil, less well compacted soil that was placed to increase the levee height, and from maintenance traffic wheel loads coming too close to the edge of the relatively narrow levee crown. As discussed previously, construction drawings indicate that the levee was designed to have an 8-foot-wide crown. Recent topographic mapping indicates that the levee crown width varies from about 9½ to 11½ feet. Increasing the levee height 2 feet and adding 2 feet of levee width could have resulted in some less compact soil conditions near the top of the levee slope as shown in Figure 15. Further, our measurements of the depth of the longitudinal cracking by probing indicate that the cracking appears to be limited to the upper 2½ feet of the levee fill.

If the observed longitudinal cracking was due to unstable slope conditions, some amount of differential movement should be readily observable, that is, the downslope side of the cracking should be lower than the upslope side of the cracking, and the cracking would be expected to have more of an arced appearance characteristic of slumping or slope failure. Except for one relatively small area between the Hidden Lakes reservoir and the railway bridge crossings, we did not observe significant differential displacement across most of the cracked areas.

6.7 Settlement

Laboratory test results indicate that both the levee fill and foundation soils are only moderately compressible. If the existing levees are not increased in height, future long term settlement of the levee is estimated to be almost nil since most of the settlement under the weight of the levee occurred within a few years of the middle 70's construction. If the existing levees are enlarged and increased in height up to 4 feet, additional settlements from the weight of the new fill are estimated to be less than 1 inch and to mostly occur during construction as the new fill weight is applied.

6.8 Erosion Protection

The creek channel is mostly covered with heavy vegetation consisting of grasses and small shrubs and the creekside slopes are mostly covered with a heavy growth of grasses and some smaller shrubs. Fairly large trees with estimated diameters of 8 to 10 inches are growing in the levee at numerous locations. Our observations indicate that the existing vegetation and ground covering appears to be protecting the creek channel and creekside slopes adequately. However, larger trees growing in the levee structural section are generally undesirable because their root systems may become pathways for seepage and because of their potential interference with levee maintenance. It is generally preferred that levee vegetation be limited to grasses and small shrubs with roots and stems smaller than about ¾-inch and root systems that do not extend more than about 2 feet below the levee surface. If larger shrubs and trees are required for landscaping, the levee section should be enlarged so that the root bulbs and systems do not extend into the levee structural section required for strength and seepage resistance.

Table 6-1. Minimum Factors of Safety – Levee Slope Stability

Type of Slope	Applicable Stability Condition and Required Factors of Safety			
	End of Construction	Long Term (Steady Seepage)	Sudden Drawdown	Earthquake
Existing levees	---	1.4	1.0	1.0
New levees	1.3	1.4	1.0	1.0

Ref.: U.S. Army Corps of Engineers (2000)

Table 6-2. Summary of Soil Properties Used for Stability Analyses

Soil	Total Density (pcf)	Undrained Shear Strength (s_u - psf)	Effective Cohesion (psf)	Effective ϕ (degrees)	Total Stress, consolidated undrained[2.]
Levee Fill	120	600	0	36	36, 500, 16
Foundation Clay	120	1,000	0	36	36, 400, 14
Foundation Sand	120	100[1.]	0	36	

[1.] Used residual strength of 100 psf for post-earthquake liquefaction analysis.

[2.] Bilinear total stress envelope for sudden drawdown analysis, zero strength at zero normal stress. Three numbers are initial ϕ, normal stress for change in ϕ, and ϕ beyond normal stress.

Table 6-3. Summary of Stability Analysis Results

Analysis Condition	Factor of Safety	Minimum Required Factor of Safety	Comments
Case I – Short[2] and Long[3] term without flooding			
Short Term			
2:1 creekside slopes	2.2 [2.3][4]	1.3	Satisfactory
1½:1 dryside slopes	2.4 [2.5]	1.3	Satisfactory
Long Term			
2:1 creekside slopes	2.1	1.4	Satisfactory
1½:1 dryside slopes	2.3	1.4	Satisfactory
Case II - Sudden Drawdown (Creekside slopes)	1.7 [2.1]	1.0	Satisfactory
Case III – Steady Seepage from Full Flood (Dryside slopes)	2.3 [1.6]	1.4	Satisfactory; seepage analysis indicates that wetting front does not advance into the levee more than several feet for flood duration of 7 days or less.
Case IV – Earthquake (0.2g) Short term without flooding			
2:1 creekside slopes	1.5	1.0	Satisfactory
1½ :1 dryside slopes	1.6	1.0	Satisfactory
Long term without flooding			
2:1 creekside slopes	1.4	1.0	Satisfactory
1½ :1 dryside slopes	1.5	1.0	Satisfactory
Liquefaction of sand layers			
Without EQ force	2.0	1.0	Satisfactory
With EQ force (0.2g)	0.8	1.0	

Notes:

1. See figures in Appendix F showing stability analysis cross sections and sliding surfaces with lowest factors of safety.
2. Short term – Undrained strengths for levee fill and foundation soils.
3. Long term – Undrained strength for levee fill; drained strength for foundation soils.
4. Values in brackets are factors of safety for similar conditions from previous analyses by others (Harza, 1998). Values in brackets are not provided where the cases analyzed were not similar.

7.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

Preliminary geotechnical recommendations for levee crown width and elevation, maximum creekside and dryside embankment slopes, and mitigating the existing longitudinal cracking are provided below.

7.1 Crown Width

While minimum levee crown widths of 10 to 12 feet are often used, these crown widths may be too narrow to avoid maintenance truck wheel loads coming too close to the edge of the crown. In general, we recommend that the levee crown width be at least 4 feet wider than the widest expected maintenance vehicle so that vehicle wheel loads do not come closer than about 2 feet to the edge of the levee crown with the vehicle centered on the levee crown.

7.2 Crown Elevation

In the past, crown elevation has generally been established to provide a height of "freeboard" above the design flood elevation to account for hydraulic, geotechnical, construction operation and maintenance uncertainties. Amounts of minimum freeboard vary from about 1 foot where the levees protect mostly uninhabited agricultural land to 2 to 3 feet where levees protect urban land. We understand that the District generally requires 3 feet of freeboard.

The U. S. Army Corps of Engineers (COE) no longer uses freeboard to account for uncertainties. In place of freeboard, the COE performs risk-based analysis to account for hydraulic uncertainties and establishes a nominal levee crown elevation. The COE then recommends deterministic analysis of the embankment and foundation materials to set the final levee grade to account for settlement, shrinkage, cracking, and construction tolerances (COE 2000).

7.3 Erosion Protection

We understand that anticipated maximum channel flow velocities would be about 4 to 6 feet per second, except where special hydraulic conditions may occur. Table 7-1 provides a range of permissible flow velocities that can be used as a guide for designing nonscouring flood control channels. The permissible flow velocities range from about 2 feet per second for fine sand and sandy silt to 20 feet per second for a sound rock. Permissible flow velocities for grass-lined earth channels range from 5 to 8 feet per second.

7.4 Maximum Slopes

Maximum slopes depend on the strength of the levee and foundation soils and whether or not the slopes are reinforced with geosynthetic materials. For ease of construction, a 2:1 slope is generally accepted as the steepest soil slope that can readily be constructed and provide stability for stone or riprap layers. A 3:1 slope is about the steepest slope that can be maintained with conventional mowing equipment and walked on for periodic inspections.

Table 7-2 shows preliminary maximum recommended levee slopes with and without geosynthetic slope reinforcement, different types of slope face protection, and approximate permissible flow velocities; Figures 18 to 20 show schematically the different types of slope facings and reinforcement, and structural walls. Table 7-3 provides a list of internet information sources for reinforced slopes and erosion protection.

As discussed previously, the existing levee slopes range from about 2.7:1 to 2:1 on the creekside and 1.8:1 to 1.4:1 on the dryside and were designed to be 2:1 on the creekside and 1½:1 on the dryside. For the most part, these slopes appear to have held up fairly well over their approximately 25-year service period. However, based on their present condition and performance, it appears that the slopes would have performed better if they were a little flatter, about 2½:1 on the creekside and 2:1 on the dryside. We base this conclusion on our observations and the amount of apparent surficial movement and cracking. Even at these slightly flatter slopes, most of the levee fill would remain susceptible to cracking due to their high to very high plasticity characteristics.

Based on the discussion above and the results of our stability analyses, we recommend maximum levee slopes for the existing levee soils of 2½:1 for the creekside slopes and 2:1 for the dryside slopes.

Our preliminary analyses indicate that maximum slopes could be increased to a little steeper than 1:1 or about 50° from horizontal with the use of geosynthetic and geogrid reinforcing materials. Steeper maximum slopes of up to 90° from horizontal could be obtained using structural retaining walls. Slopes steeper than about 60° to 70° are usually designed as structural walls.

In summary, maximum levee slopes could range from about 2½:1 to 2:1 for unreinforced soils slopes to about 50° for reinforced soil slopes, and could range up to 90° from horizontal using structural walls.

Slope facings most conducive to landscaping would be soil slopes, reinforced soil slopes with an erosion control mat, or structural walls with openings or benches for planting. Depending on the type and size of the landscaping, the levee section beneath the zones of landscaping and root systems must be able to satisfy all of the requirements for stability, seepage resistance, and settlement. This may require enlarging the levee section to provide sufficient space for the landscaping and satisfy the required levee design criteria.

7.5 Existing Levee Longitudinal Cracking

As discussed previously, the existing levee cracking does not appear to be due to levee slope failures or larger slope movements, except in a few areas. These areas are about a 200 foot section of the east side levee south of the railway bridges and north of the Hidden Hills detention reservoir, and, possibly shallow surficial instability for about 2,000 feet of the east levee south of the Hidden Hills detention reservoir and north of Milpitas Boulevard. We understand that these areas have been landscaped and have sprinkler systems. Our experience has been that landscaping and watering steeper slopes of about 1½:1 often results in excessive surficial creep due to softening of the surficial soils and the steepness of the slopes. Some alternatives for managing these conditions would be to discontinue the landscape sprinkling, change the type of landscaping to minimize watering requirements and to flatten the slopes.

Even though the observed levee cracking does not appear to affect levee stability or seepage resistance, we recommend that the cracking be mitigated by the temporary measures of filling the cracks, or by removing or partly removing the top approximately 3 feet of existing soils in the cracked areas and replacing the removed soils with suitable lower plasticity soils. Preliminary earthwork construction recommendations are provided in a following section.

The cracks could be filled with a compacted mixture of moist sand and about 8 percent bentonite. Before filling, the cracks should be prepared for filling by slightly enlarging and cleaning them with a small, narrow ditching machine. Then, the cracks should be incrementally filled and tamped with a specially made steel tamping rod or other effective tamping method. Actual proportions for the sand-bentonite mixture should be based on preparing laboratory trial mixes using samples of the proposed materials.

7.6 Landscaping Considerations

As discussed in the previous section, it is generally preferred that levee vegetation and planting be limited to grasses and small shrubs with roots and stems smaller than about ¾-inch and root systems that do not extend more than about a foot below the levee surface. If larger shrubs and trees are required for landscaping, the levee section should be enlarged so that the root bulbs and systems do not extend into the levee structural section required for strength and seepage resistance.

7.7 Earthwork Construction

Preliminary earthwork construction recommendations are provided below. These preliminary recommendations should be reviewed and revised as appropriate for the design construction.

7.7.1 Clearing and Site Preparation

Prior to fill placement, the site, to a distance at least 10 feet outside the work areas, should be cleared of all surface and subsurface improvements to be removed and all deleterious materials including existing irrigation lines, fills, pavements, debris, designated trees, shrubs, and associated roots. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report.

After clearing and removal of all deleterious materials, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 2 to 4 inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

7.7.2 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

7.7.3 Material for Fill

Fill material for use in the levees at the site should be onsite levee fill and foundation soils or imported fill conforming with the following requirements for fill. The fill materials should not contain rocks or lumps larger than 4 inches in greatest dimension, with no more than 15 percent larger than 2½ inches. At least 75 percent of the fill should have a particle size passing the No. 4 sieve with no less than 50 percent passing the No. 200 sieve. Fill material should have a Liquid Limit less than 40 percent and a Plasticity Index between 10 and 20. After compaction, the fill should have a hydraulic conductivity less than 1×10^{-6} cm/sec. A sample of all proposed imported fill or alternate borrow sources should be submitted for compliance testing at least two weeks prior to the proposed importing date. Consideration should also be given to the environmental characteristics of imported soils.

7.7.4 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, using appropriate size steel tamping feet.

Fill should be placed in lifts no greater than 8 inches in uncompacted thickness at moisture content at least 3 percent over the laboratory optimum. Each successive lift should be firm and non-yielding under the weight of construction equipment. Fill surfaces that become too smooth or dry to suitably bond with succeeding layers shall be moistened and scarified. Backfill within 5 feet of retaining walls and other concrete structures should be compacted by hand-operated compacting equipment. The field density and moisture content of compacted fill should be determined according to ASTM D2922 and D3017 standard methods. Any backfilled areas not meeting the minimum test requirements should be removed and recompacted until tests meet or exceed the minimum requirements. Water jetting or ponding in an attempt to obtain compaction should not be allowed.

7.7.5 Wet Weather Conditions

Earthwork contractors should be made aware of the moisture sensitivity of clayey soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft and unworkable. Typical stabilization techniques such as chemical treatment or the use of stabilization fabrics and/or crushed rock may not be applicable for the construction of levees. Aeration and reconditioning would most likely be required to stabilize wet or saturated unworkable soils. Therefore, we recommend that consideration be given to construction during summer months.

Table 7-1. Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity (feet/sec)
Fine sand	2
Coarse sand	4
Fine gravel	6
Earth	
Sandy silt	2
Silty clay	3½
Clay	6
Grass-lined Earth	
Bermuda grass	
Sandy silt	6
Silty clay	8
Kentucky Blue grass	
Sandy silt	5
Silty clay	7
Poor Rock (usually sedimentary)	
Soft sandstone	10
Soft shale	3½
Good Rock (usually igneous or hard metamorphic)	20

Reference: COE 1991

Table 7-2. Summary of Slope Facings, Maximum Slopes, and Permissible Velocities

Slope Facing	Maximum Slope	Approx. Permissable Velocity	Land-scaping Ease[1]	Comments
Soil Slopes				
None	2½:1, floodside 2:1, dryside	2	1	
Grasses, small shrubs	2½:1, floodside 2:1, dryside	4 – 6	1	
Riprap	2½:1, floodside 2:1, dryside	10	3	Difficult to establish vegetation on riprap slopes.
Articulated block mat	2½:1, floodside 2:1, dryside	6 – 12	2	Less conducive to landscaping.
Concrete lining	2½:1, floodside 2:1, dryside	>12	5	Not conducive to landscaping.
Lime or cement treatment	1:1	>8	3	Difficult to establish vegetation on steep cemented slopes.
Reinforced Soil Slopes				
Geosynthetic wrapped face	1:1	6 - 10	2	Geosynthetic materials subject to long-term degradation and vandalism.
Erosion control mats	1:1	6 – 10	2	Erosion control mat subject to long-term degradation and vandalism.
Gabions	1:1	6 – 12	3	Gabion basket materials subject to long-term degradation and vandalism.
Structural Walls				
Gabions	½:1	6 – 12	3	Gabion basket materials subject to long-term degradation and vandalism.
Mechanically stabilized earth wall	Vertical	> 12	3	
Crib wall	½:1	>12	3	
Concrete wall	Vertical	>12	5	Not conducive to landscaping.

Notes:

1. Numerical ranking ranging from 1 for relatively easy to 5 for difficult or impractical.

Table 7-3. List of Reinforced Slope Information Sources

MANUFACTURER	SYSTEM	WEB ADDRESS	COMMENTS
Tensar	Wire baskets and geogrid reinforcing	www.tensarcorp.com	Look under Systems\Sierra Slope Retention System
Maccaferri	Wire baskets and geogrid reinforcing	www.maccaferri-usa.com	Look under Products\Green Terramesh and Gabions. Also look under case studies and project examples
TC Mirafi	Geogrid reinforcing	www.tcmirafi.com	Look under reinforced soil, steepened slopes, geosynthetics in use
Verdura	Segmental concrete blocks with geogrid reinforcing	www.soilretention.com	Look under products and trade papers
Presto	Geocells	www.prestogeo.com	Look under slope protection systems, earth retention systems, and case studies
Armortec	Articulated concrete block	www.armortec.com	Look under Armortec solutions
Contech Construction Products	Geogrids, erosion control mats	www.contech-cpi.com	Look under geogrids and erosion control
Nilex	Geogrids	www.nilex.com	Look under applications, products and concepts
Synthetic Industries	Turf reinforcement mats, erosion control blankets	www.fixsoil.com	Look under products, Pyramat
International Erosion Control Association		www.ieca.org	Wide range of erosion control and retaining wall information

8.0 LIMITATIONS

This report has been prepared for the sole use of Santa Clara Valley Water District, specifically for design of the Berryessa Creek Levees Project in Milpitas, California. The opinions presented in this report have been formulated in accordance with generally accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was prepared. No other warranty, expressed or implied, is made or should be inferred. We are not responsible for data presented by others.

The opinions, conclusions and recommendations contained in this report are based upon information obtained from explorations at widely separated locations, site reconnaissance, review of data made available to us, and upon local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. In addition, geotechnical issues may arise that are not apparent at this time.

The geotechnical engineer should be retained to review final plans and specifications when they are available to verify these documents are consistent with the intent of the geotechnical recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction in order to evaluate compliance with our recommendations. If we are not retained for these services, Lowney Associates cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Lowney Associates' report by others. Furthermore, Lowney Associates will cease to be the Geotechnical-Engineer-of-Record at the time another consultant is retained for follow-up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any property other than that evaluated.

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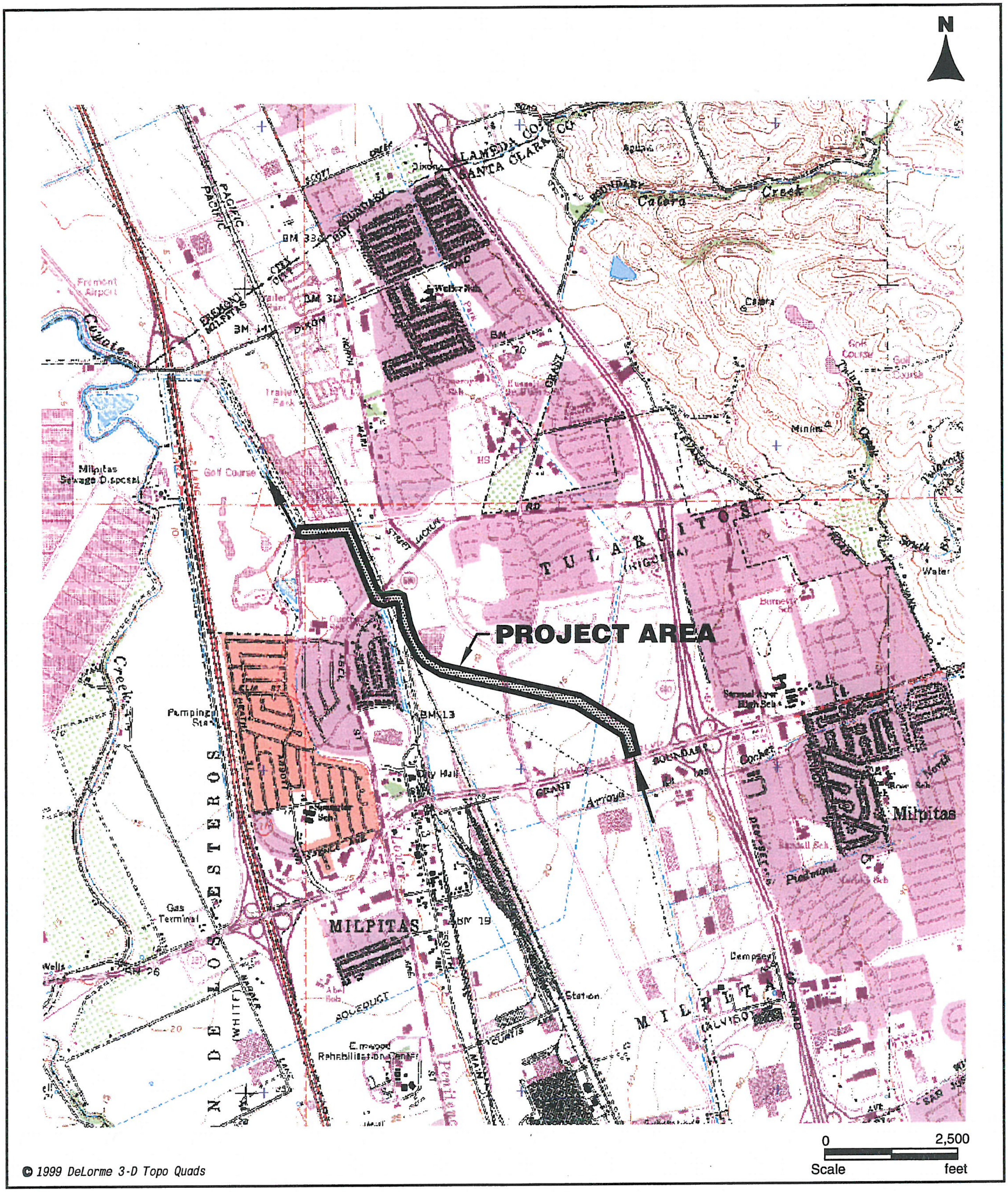

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VICINITY MAP

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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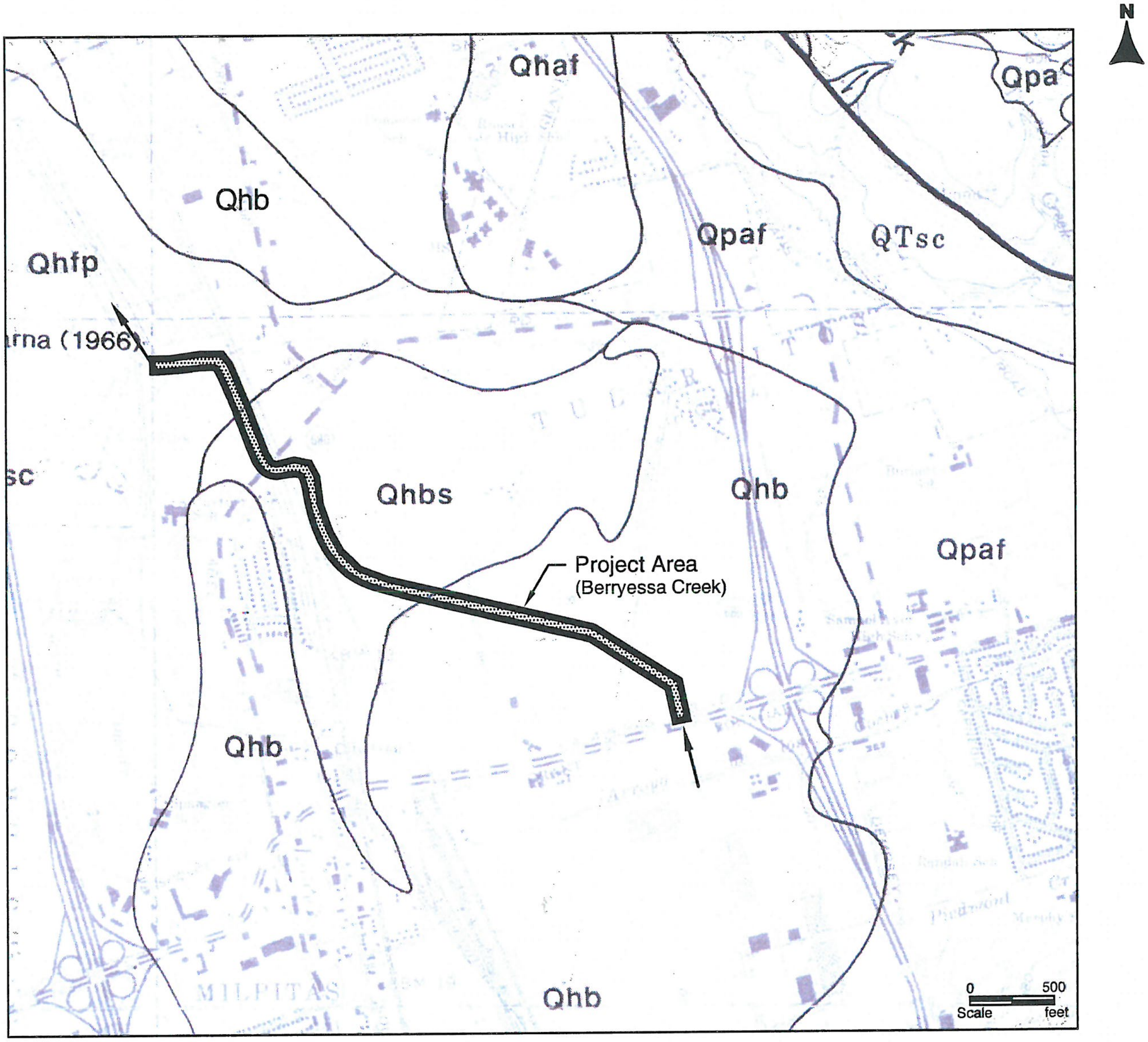
FIGURE 1
1697-1



GEOLOGIC EXPLANATION

Qhaf	ALLUVIAL FAN DEPOSITS (Holocene) Near the distal fan edges (as in the Milpitas quadrange), deposits are typically brown, medium dense, gravelly sand or clayey gravel that grade upward to sandy or silty clay
Qhb	FLOODBASIN DEPOSITS (Holocene) Organic rich clay to very fine silty clay deposits occupying the lowest topographic positions either between the Holocene levee deposits or Holocene floodplain deposits
Qhbs	FLOODBASIN DEPOSITS (Salt-affected) (Holocene) Clay to very fine silty clay deposits similar to the Qhb deposits except that they contain carbonate nodules and iron-stained mottles (Soil Conservation Service 1958)
Qhfp	FLOODPLAIN DEPOSITS (Holocene) Medium to dark gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand and pebbles) may be locally present. Floodplain deposits are found between levee deposits of Coyote Creek and Guadalupe River
Qpaf	ALLUVIAL FAN DEPOSITS (Pleistocene) Tan to reddish brown, dense, gravelly and clayey sand or clayey gravel that grades upward to a sandy clay
QTsc	SANTA CLARA FORMATION (Pliocene and Pleistocene) (Crittenden, 1951) The unconsolidated sediments of the Santa Clara Formation crop out almost continuously along the northeast corner of the Santa Clara Valley. Where exposed, the Santa Clara Formation consists of poorly-sorted gravel, pebbly sandstone, siltstone, and clay

Base by USGS Quaternary Geologic Map by E.J. Helley and J.R. Wesling dated 1989.

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GEOLOGY MAP

BERRYESSA CREEK LEVEES PROJECT

Milpitas, California

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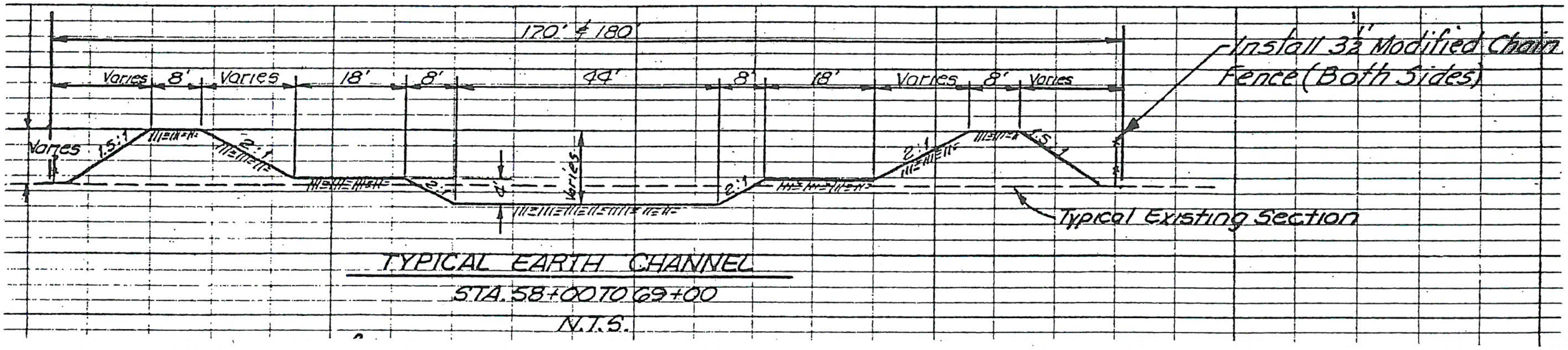
FIGURE 2
1697-1



Ref: SCVWD, 1976.

Not To Scale

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TYPICAL EARTH CHANNEL (Original Construction)

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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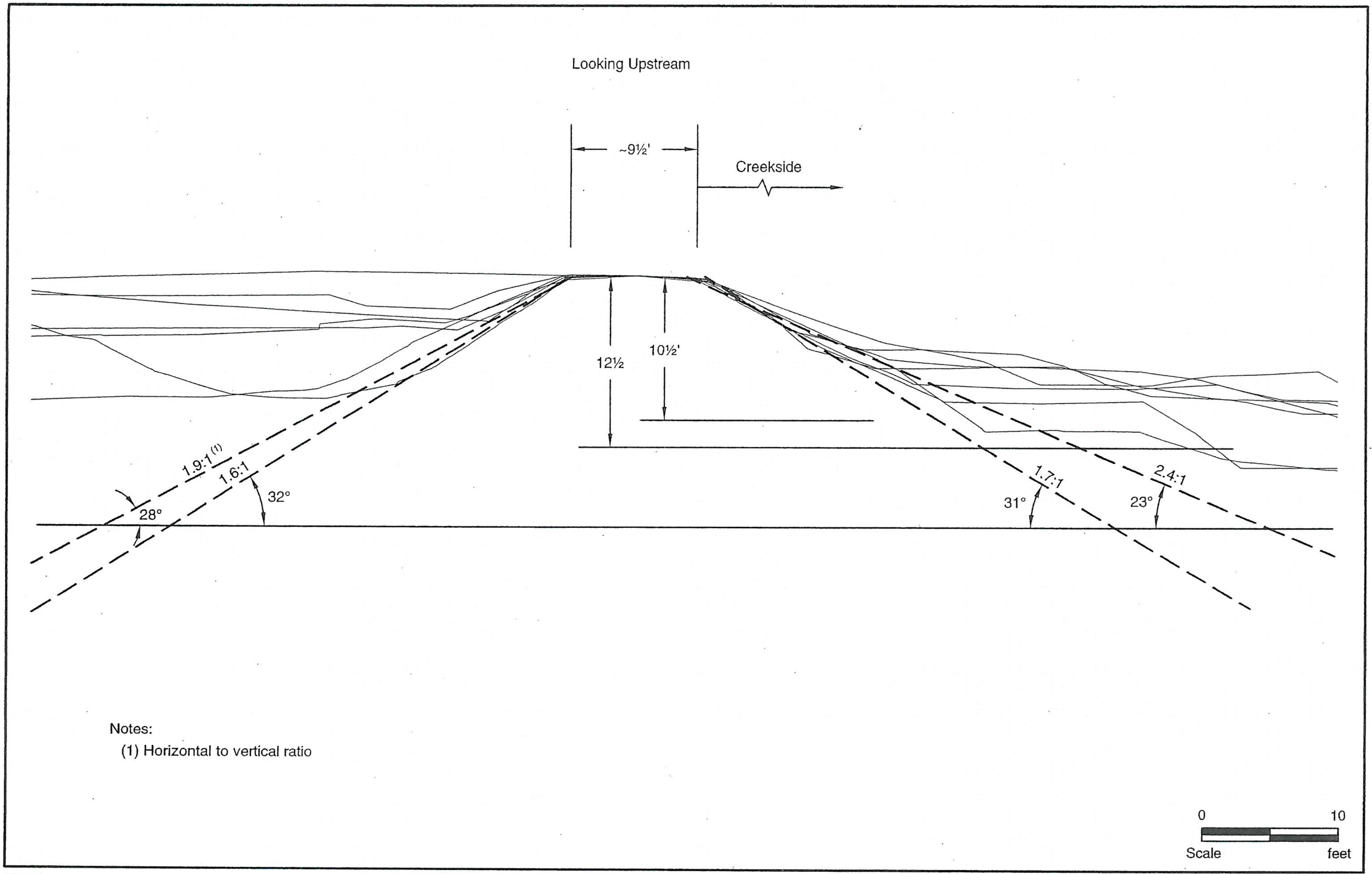
FIGURE 3
1697-1



Notes:
(1) Horizontal to vertical ratio

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EXISTING LEVEE CROSS SECTIONS (RIGHT SIDE)
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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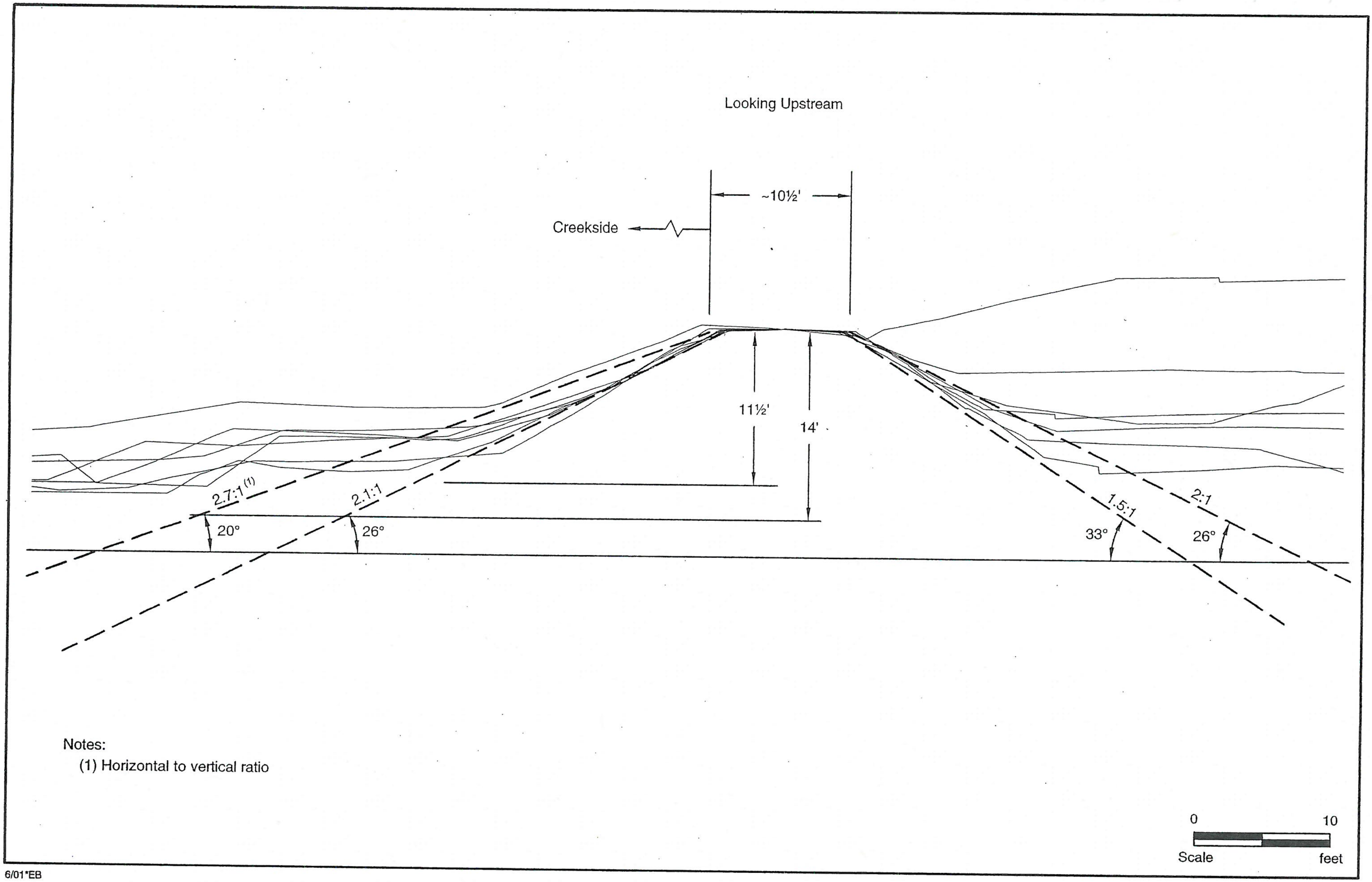
FIGURE 4
1697-1



Notes:
(1) Horizontal to vertical ratio

6/01*EB

EXISTING LEVEE CROSS SECTIONS (LEFT SIDE)

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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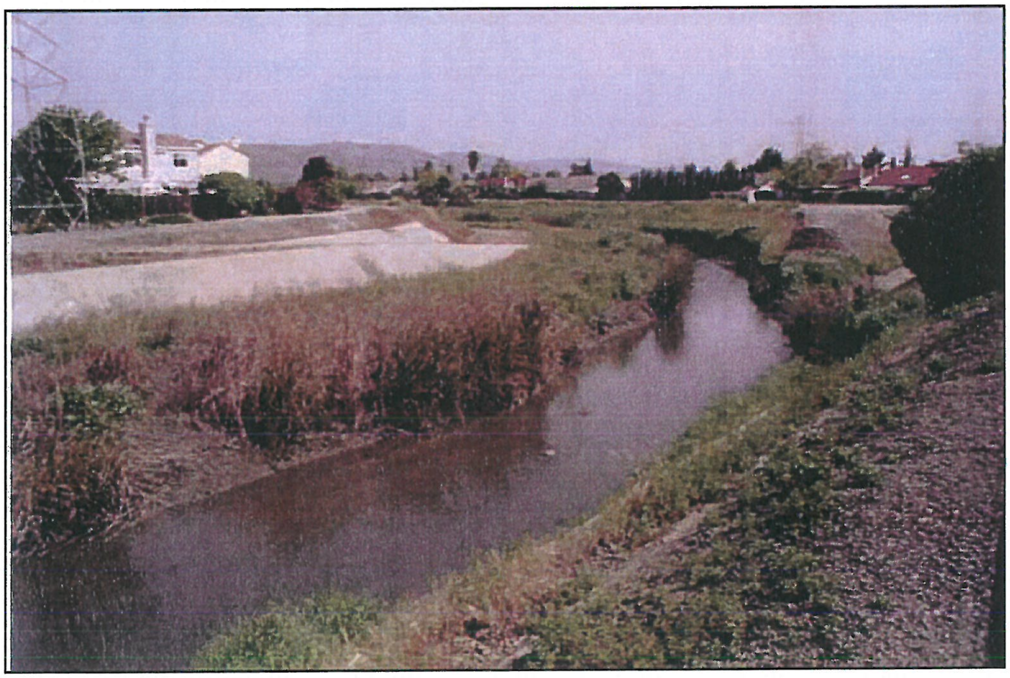
FIGURE 5
1697-1

PHOTOGRAPH 1
View easterly, near project beginning



PHOTOGRAPH 2
Confluence Berryessa and Lower Penetencia Creek

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SITE PHOTOGRAPHS 1 AND 2

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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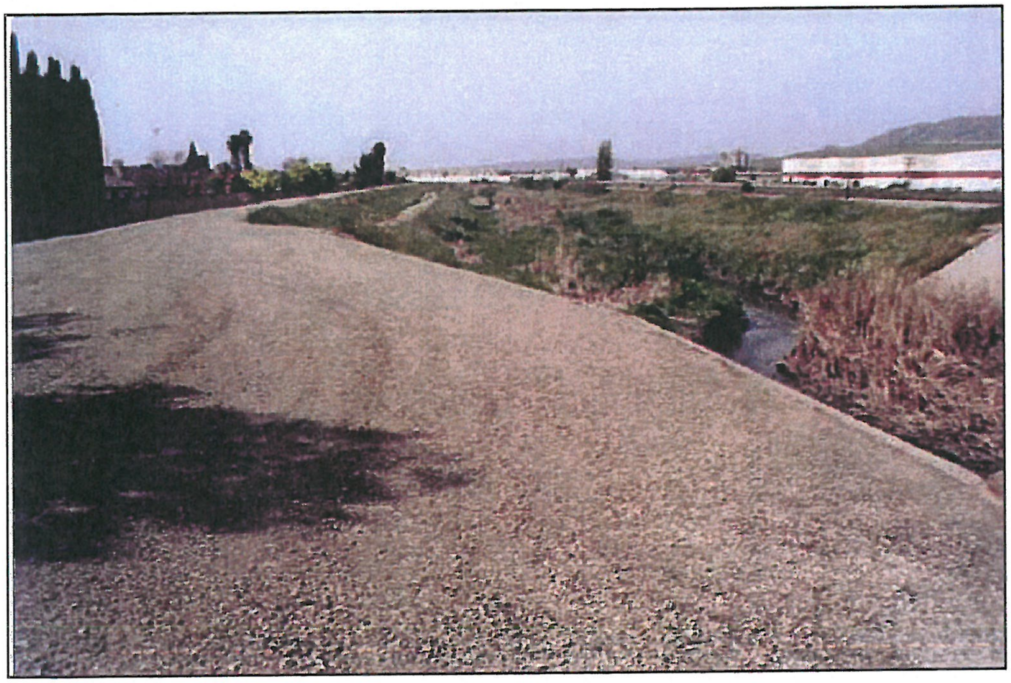
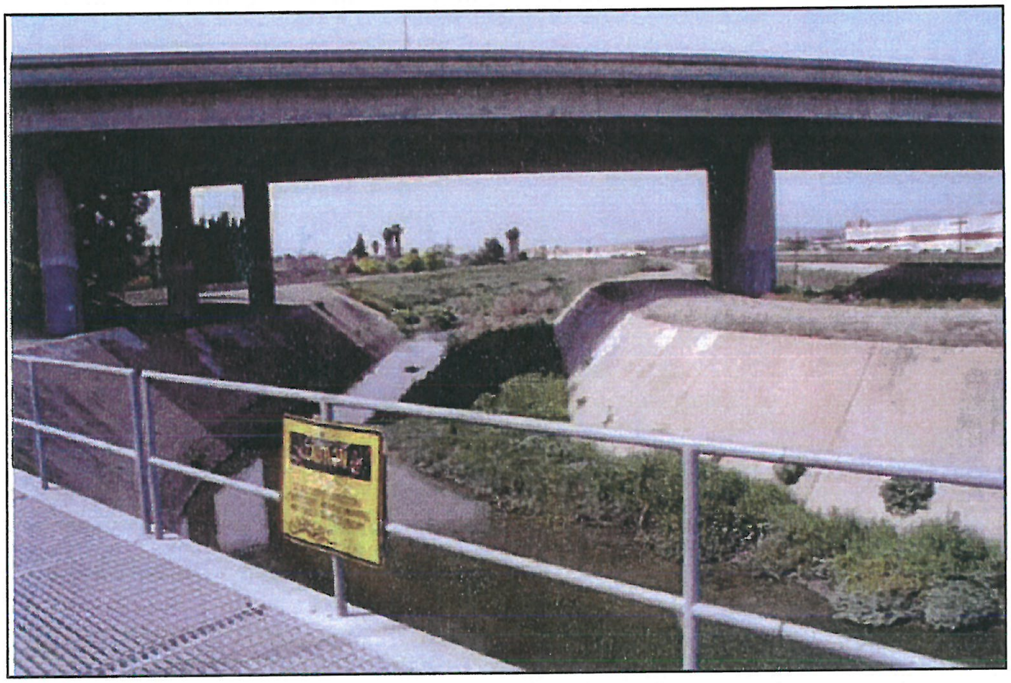
FIGURE 6
1697-1



PHOTOGRAPH 3
View northerly, near Abel Street Bridge



PHOTOGRAPH 4
View northerly, a little upstream form Abel Street Bridge

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SITE PHOTOGRAPHS 3 AND 4

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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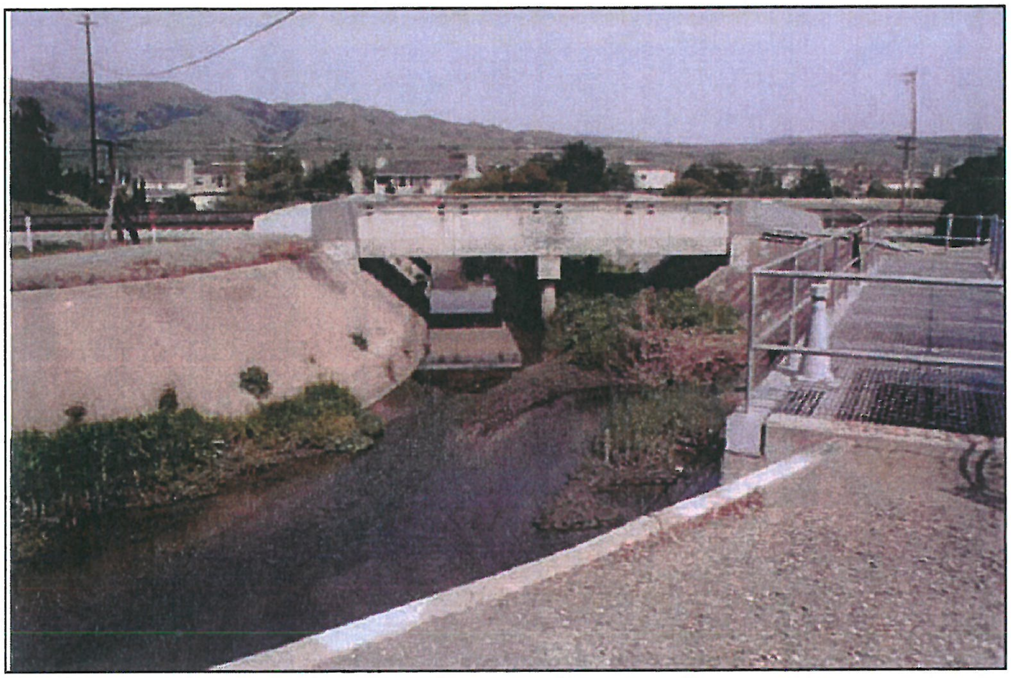
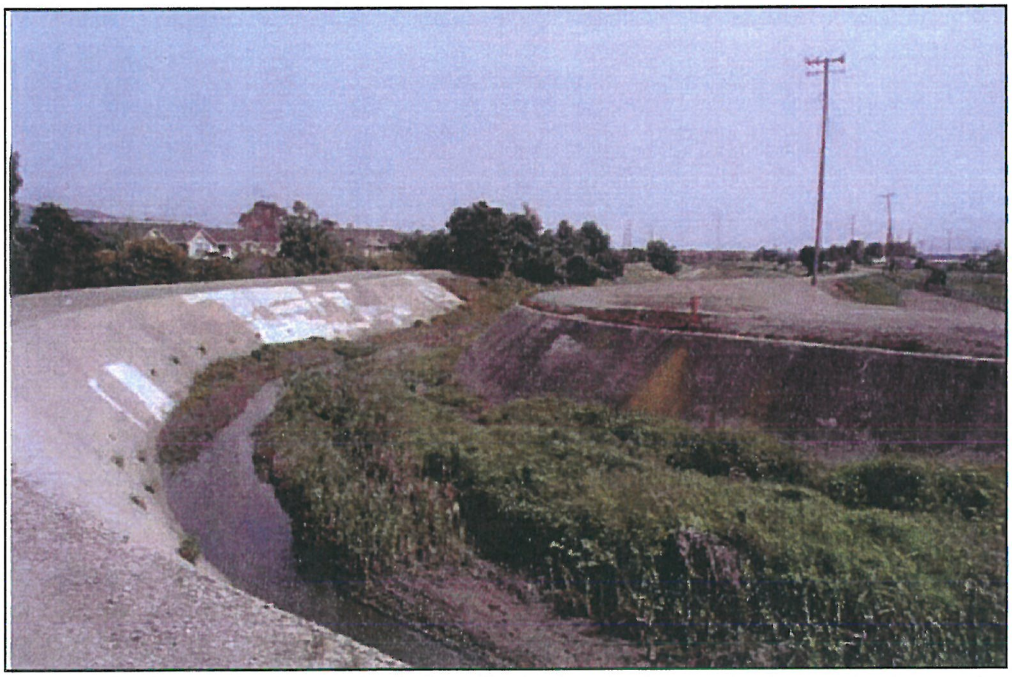
FIGURE 7
1697-1



PHOTOGRAPH 5
View easterly near railroad crossings



PHOTOGRAPH 6
View southerly, just beyond railroad crossings

3/01*EB

SITE PHOTOGRAPHS 5 AND 6

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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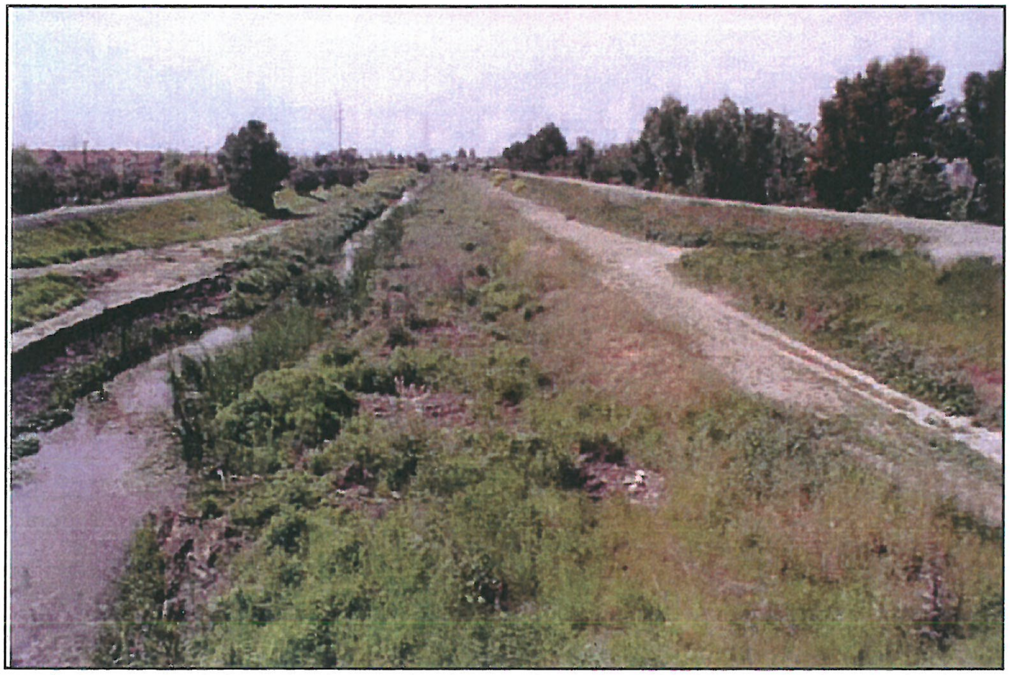
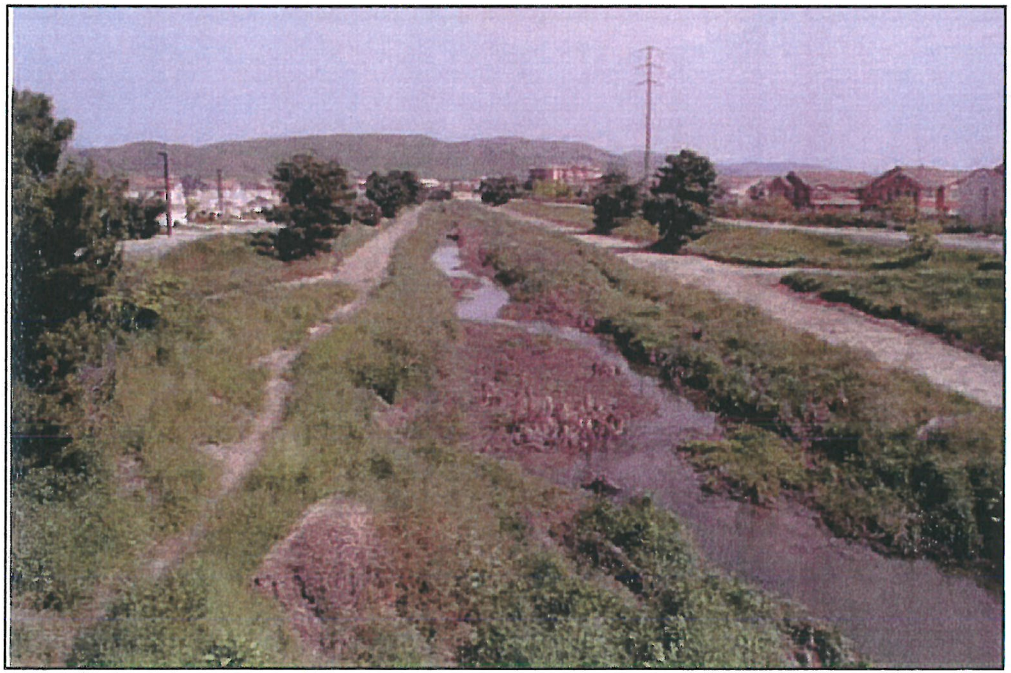
FIGURE 8
1697-1



PHOTOGRAPH 7
View northerly from Milpitas Boulevard Bridge



PHOTOGRAPH 8
View southerly from Milpitas Boulevard Bridge

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SITE PHOTOGRAPHS 7 AND 8

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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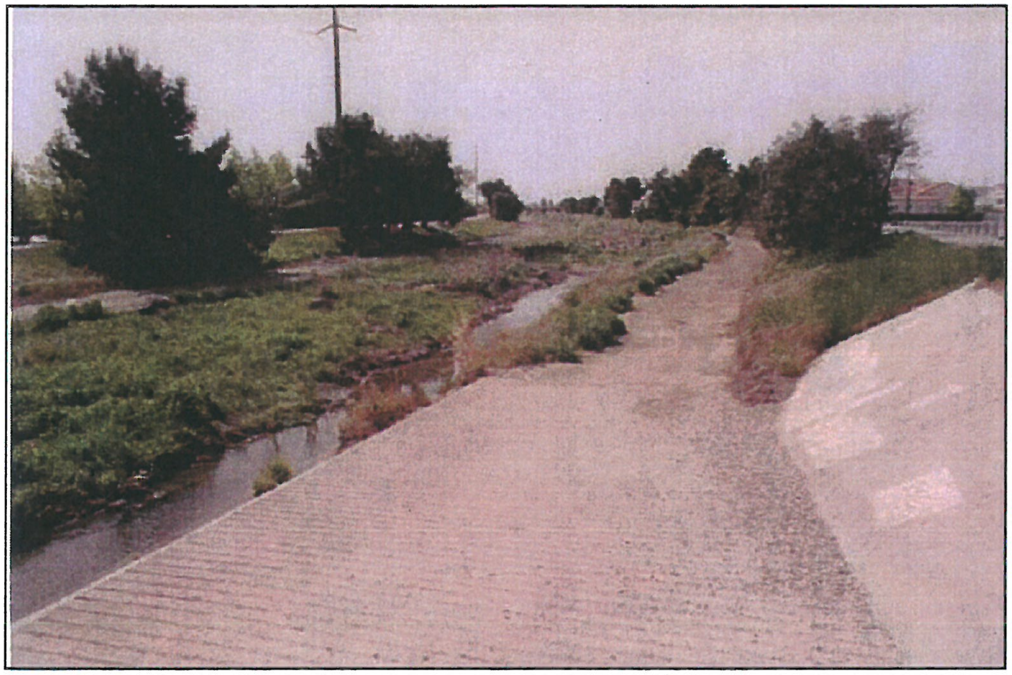
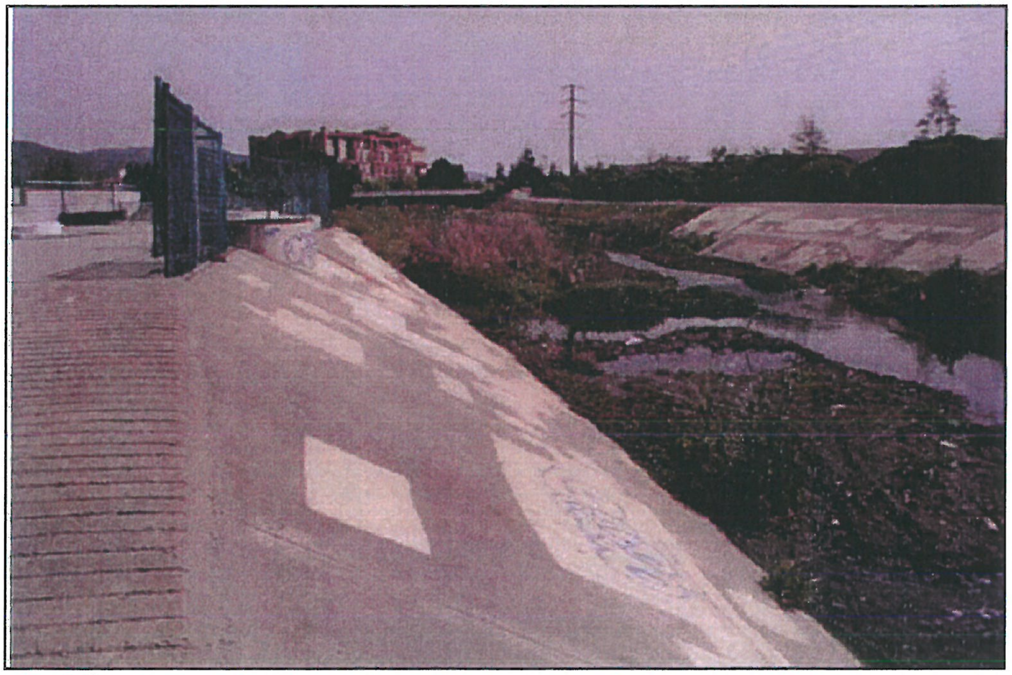
FIGURE 9
1697-1



PHOTOGRAPH 9
View northerly, near Hetch Hetchy pipeline crossing



PHOTOGRAPH 10
View southerly, near Hetch Hetchy pipline crossing

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SITE PHOTOGRAPHS 9 AND 10

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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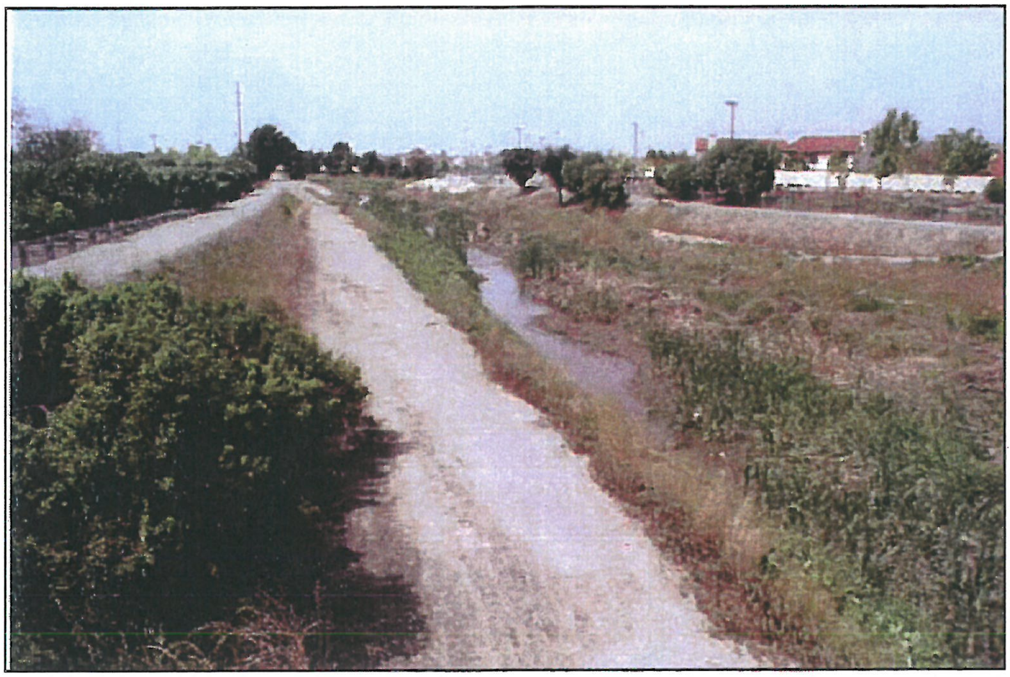
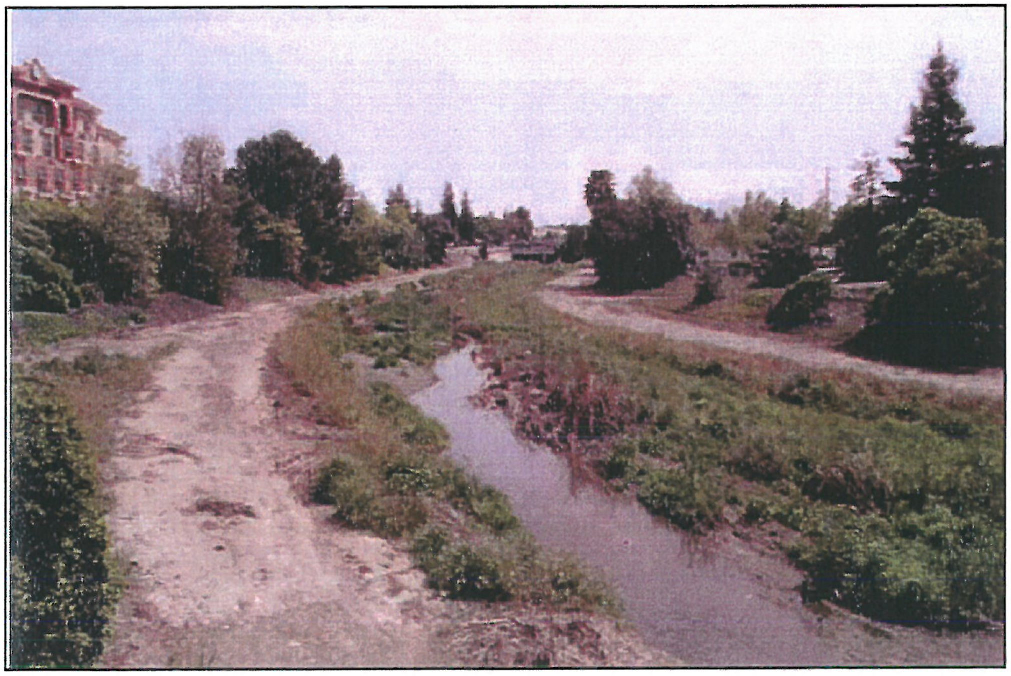
FIGURE 10
1697-1



PHOTOGRAPH 11
View northerly, from Hillview Avenue bridge



PHOTOGRAPH 12
View southerly, from Hillview Avenue bridge

3/01*EB

SITE PHOTOGRAPHS 11 AND 12

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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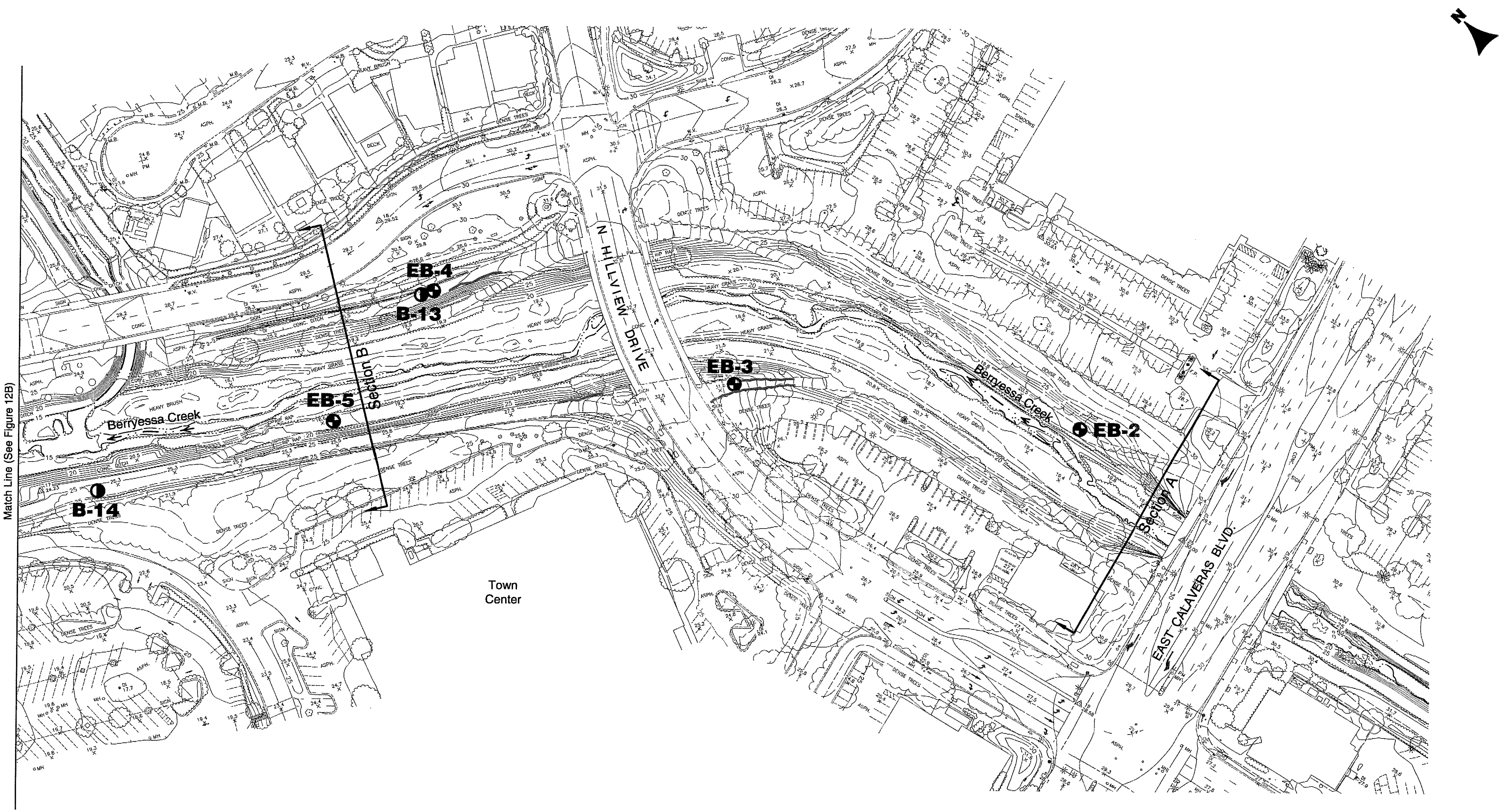
FIGURE 11
1697-1



LEGEND

- ● - Approximate location of exploratory boring by Lowney Associates (4/01-5/01)
- ◐ - Approximate location of exploratory boring by SCVWD (10/97)
- ～～ - Surface cracking

Base by Winzler & Kelly

6/01*EB

0 100
Scale feet

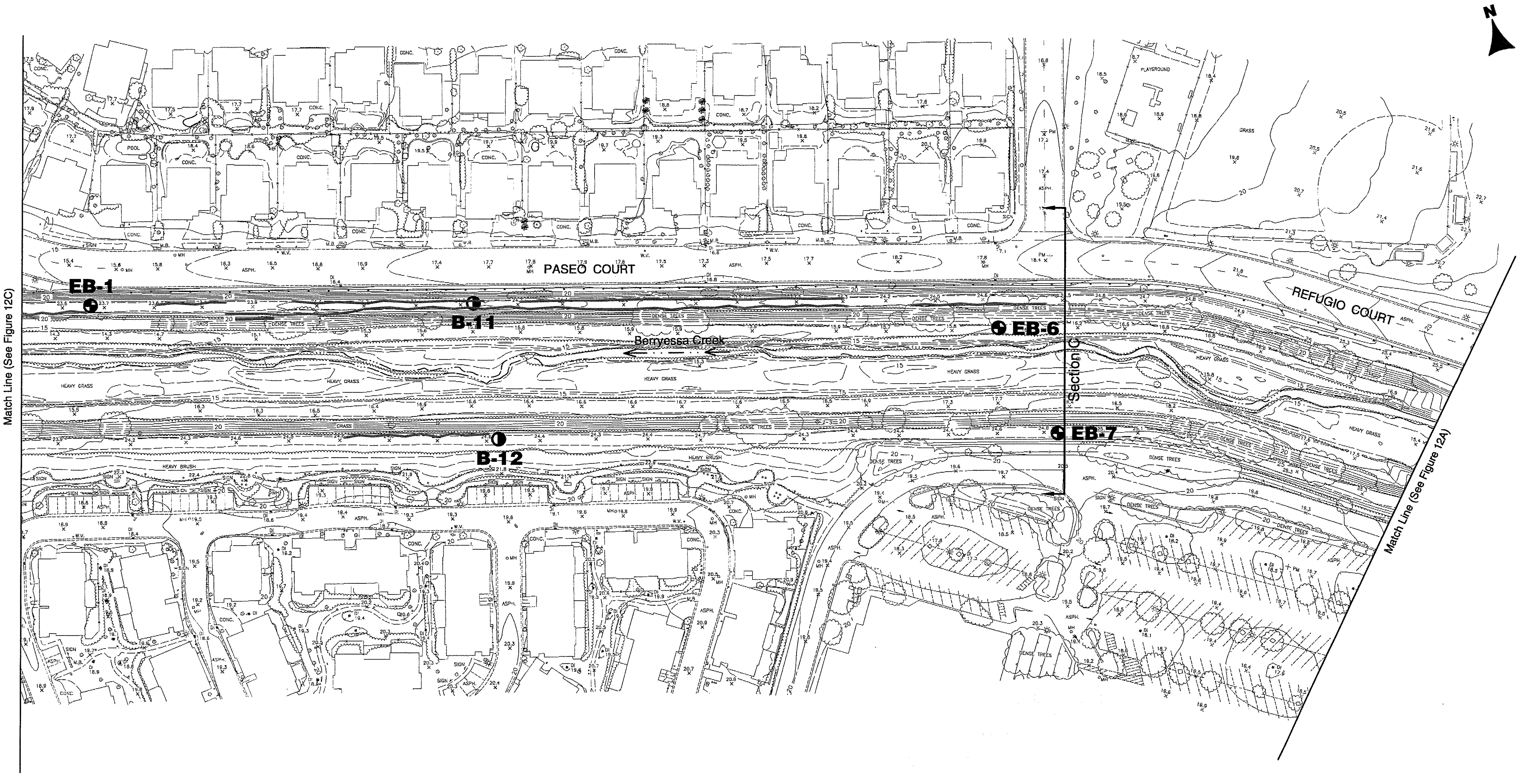
SITE PLAN

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 12A

1697-1



LEGEND

- ⊕ - Approximate location of exploratory boring by Lowney Associates (4/01-5/01)
- ◑ - Approximate location of exploratory boring by SCVWD (10/97)
- 〰 - Surface cracking

0 100
Scale feet

Base by Winzler & Kelly

SITE PLAN

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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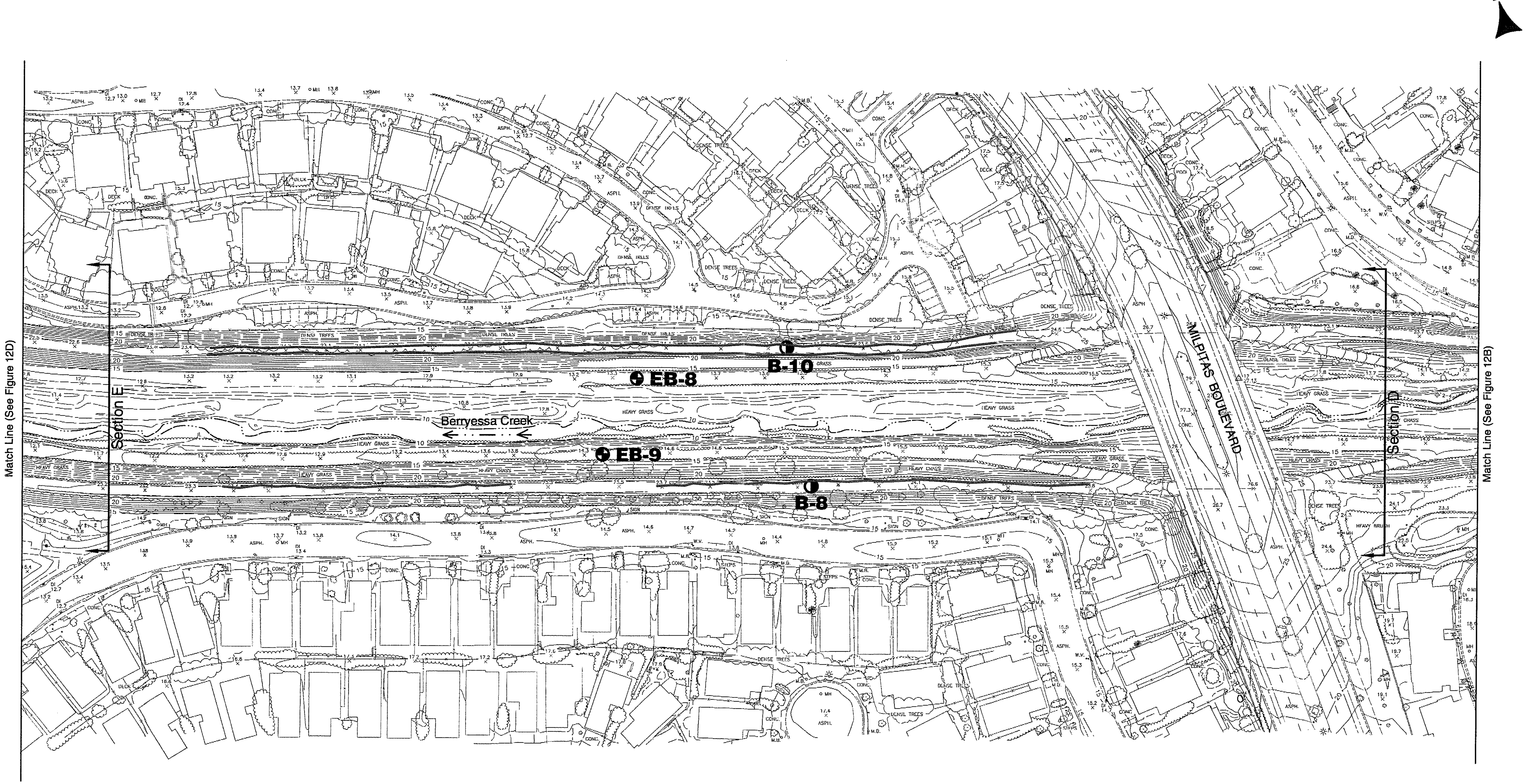
Environmental/Geotechnical/Engineering Services

FIGURE 12B

1697-1

6/01*EB



LEGEND

- ⊕ - Approximate location of exploratory boring by Lowney Associates (4/01-5/01)
- ◐ - Approximate location of exploratory boring by SCVWD (10/97)
- ~~~~ - Surface cracking

Base by Winzler & Kelly

0 100

Scale feet

SITE PLAN

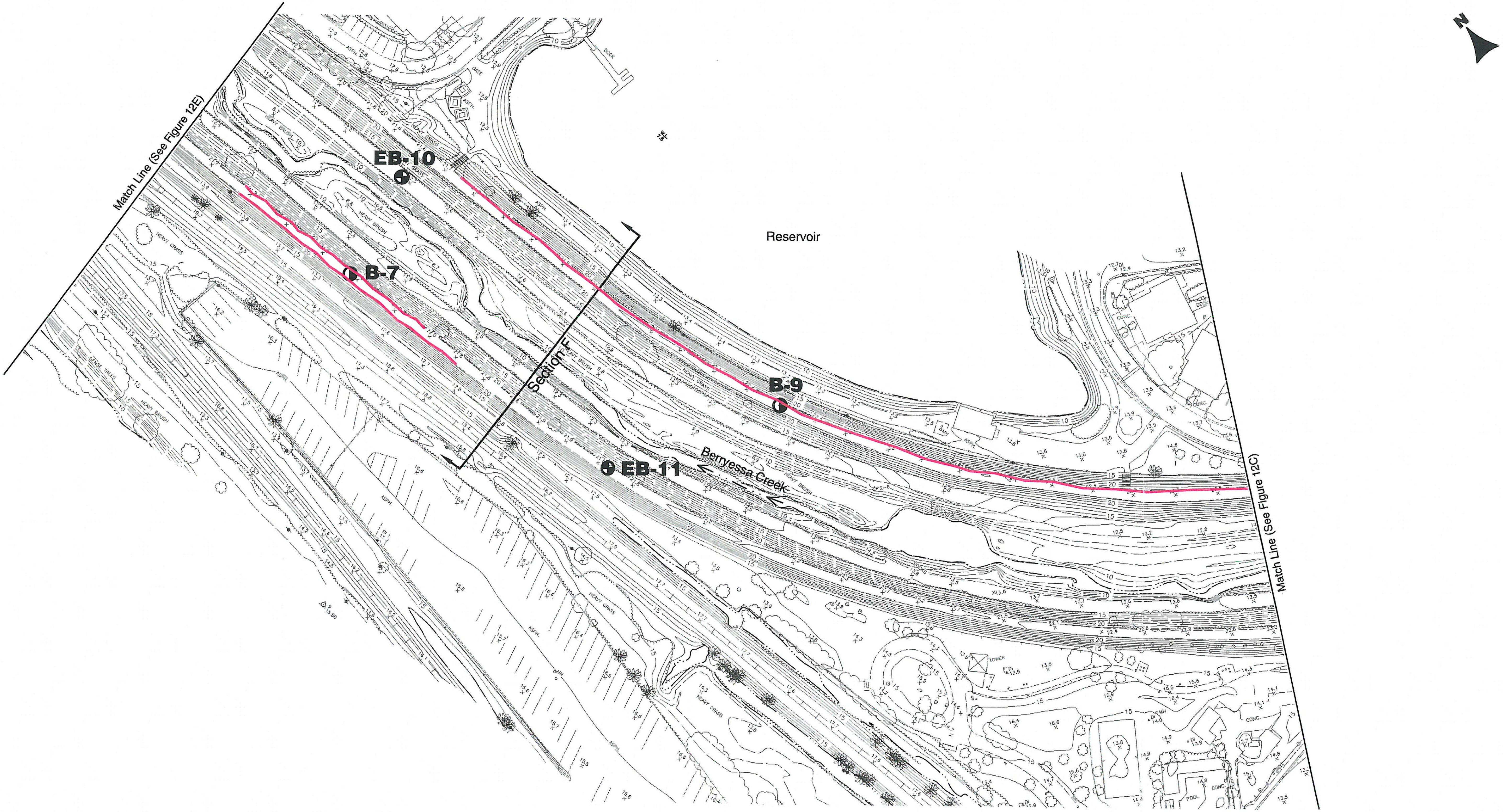
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 12C

1697-1

6/01*EB



LEGEND

- ⊕ - Approximate location of exploratory boring by Lowney Associates (4/01-5/01)
- ◐ - Approximate location of exploratory boring by SCVWD (10/97)
- 〰 - Surface cracking

Base by Winzler & Kelly

6/01*EB

0 100
Scale feet

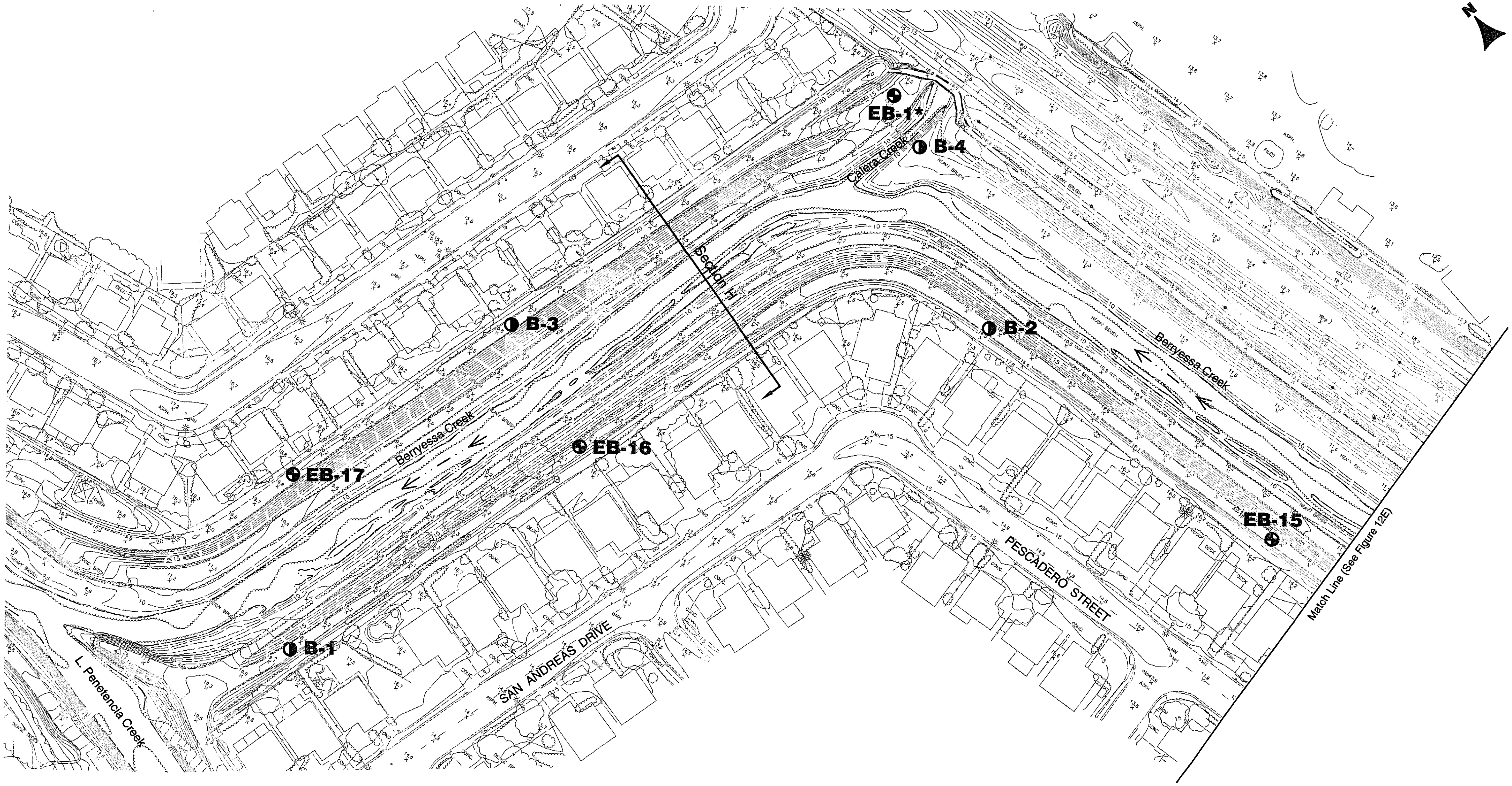
SITE PLAN

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 12D

1697-1



LEGEND

- ⊕ - Approximate location of exploratory boring by Lowney Associates (4/01-5/01)
- ◐ - Approximate location of exploratory boring by SCVWD (10/97)

*From Calera Creek Access Bridge Project

Base by Winzler & Kelly

6/01*EB

0 100
Scale feet

SITE PLAN

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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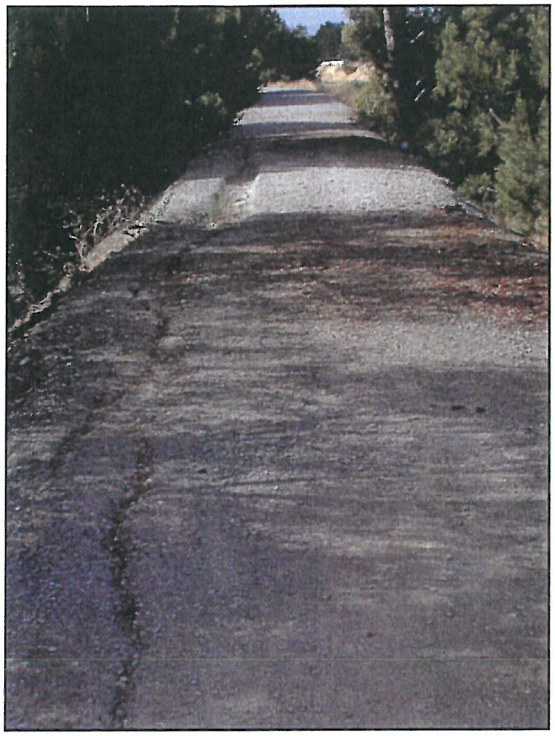
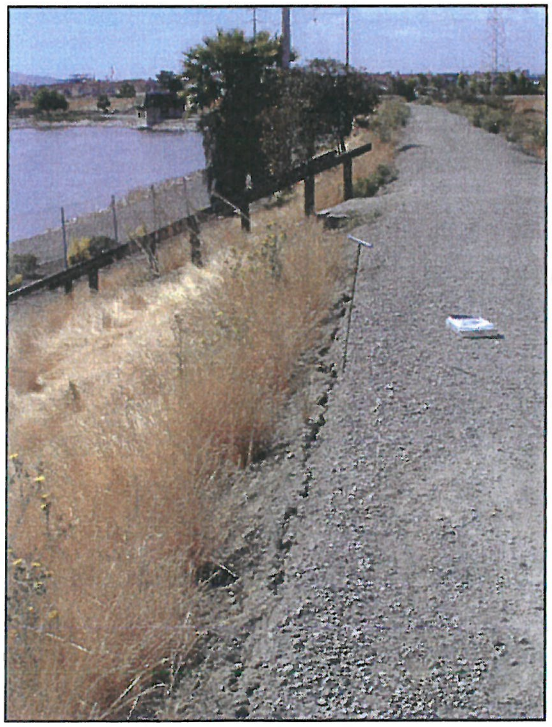
Environmental/Geotechnical/Engineering Services

FIGURE 12F

1697-1



PHOTOGRAPH 13
Sloughed and cracked area south of railway bridge



PHOTOGRAPH 14
Cracking near Hidden Lakes Reservoir

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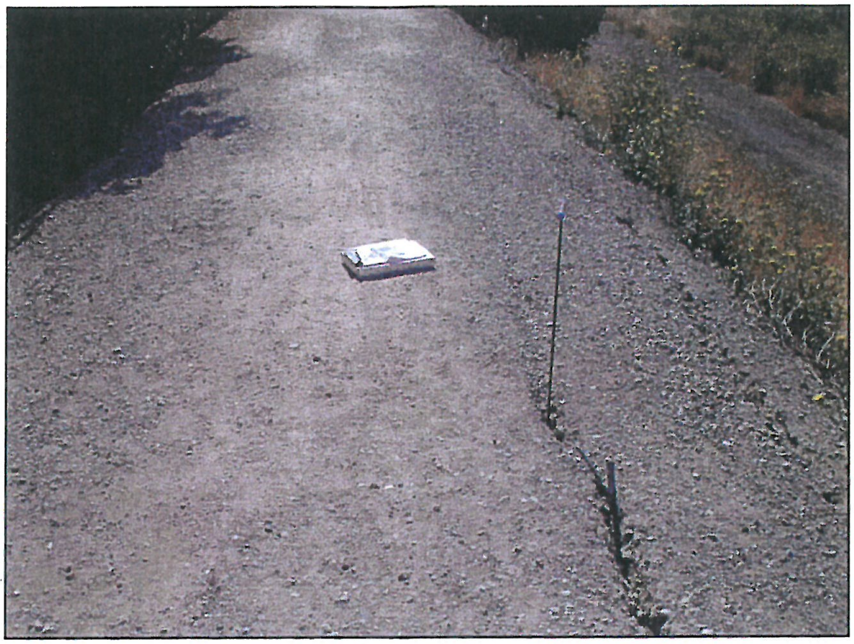
CRACKED AREA PHOTOGRAPHS

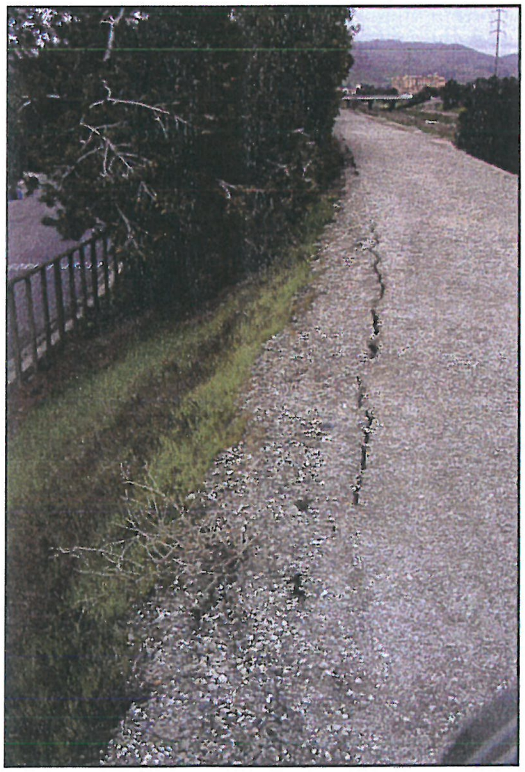
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 13
1697-1



PHOTOGRAPH 15
Cracking near Hidden Lakes Reservoir



PHOTOGRAPH 16
Cracking between Hidden Lakes Reservoir and Milpitas Boulevard

6/01*EB

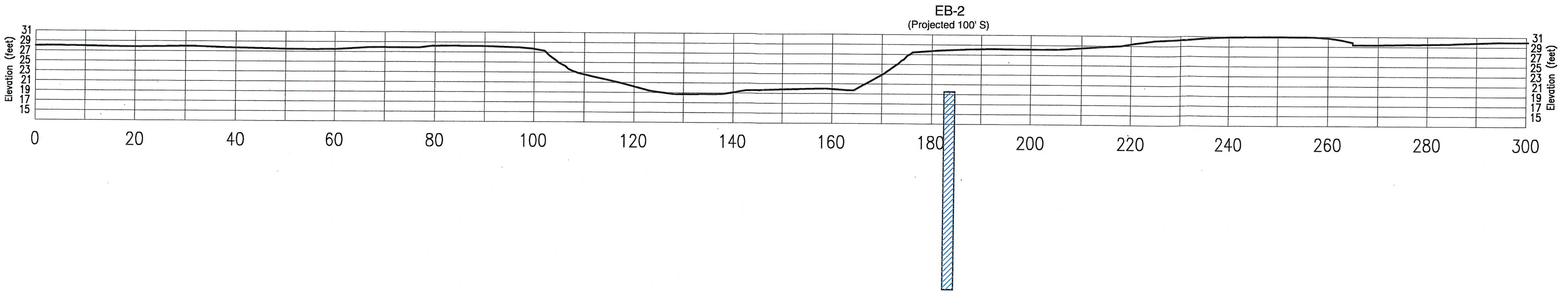
CRACKED AREA PHOTOGRAPHS

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

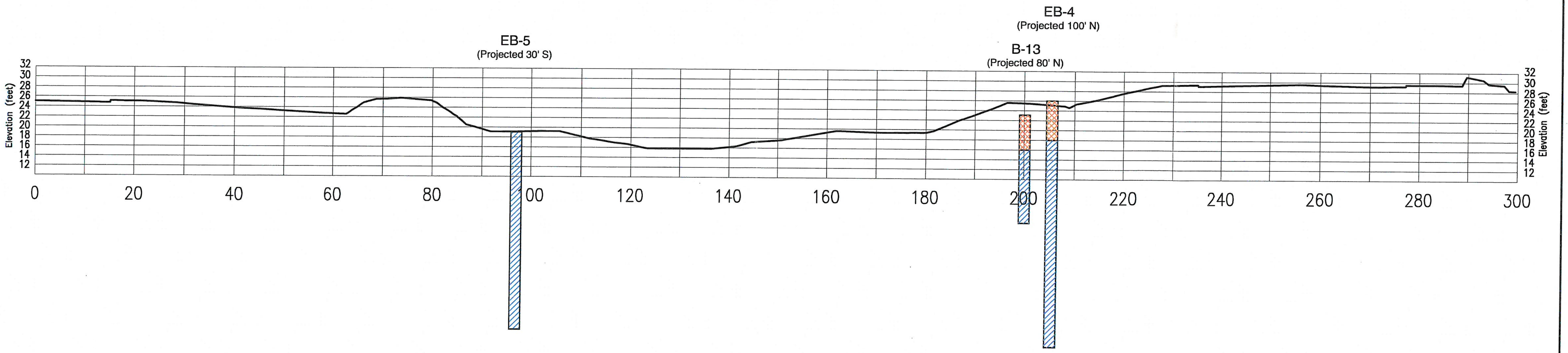
LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 14
1697-1

SECTION A



SECTION B



LEGEND

▨	Levee Fill	Mostly very high plasticity clays, medium stiff to stiff, dark gray, dark brown and gray, moist
■	Native	
▨	Clay-Silt Found. Soils	Mostly high plasticity silty clays with sand, medium stiff to stiff, light brown to brown, moist to wet
▒	Sand-Silt Found. Soils	Mostly sands and silty sands, medium dense, gray, wet

Notes:

1.) Soil profile interpreted and generalized form boring data; conditions may be different away from the boring locations and between sample depths at boring locations.

2.) Refer to boring logs for more detailed information.

3.) Cross section locations shown on Figures 12A to 12F; cross sections are looking downstream

0 20
Scale feet

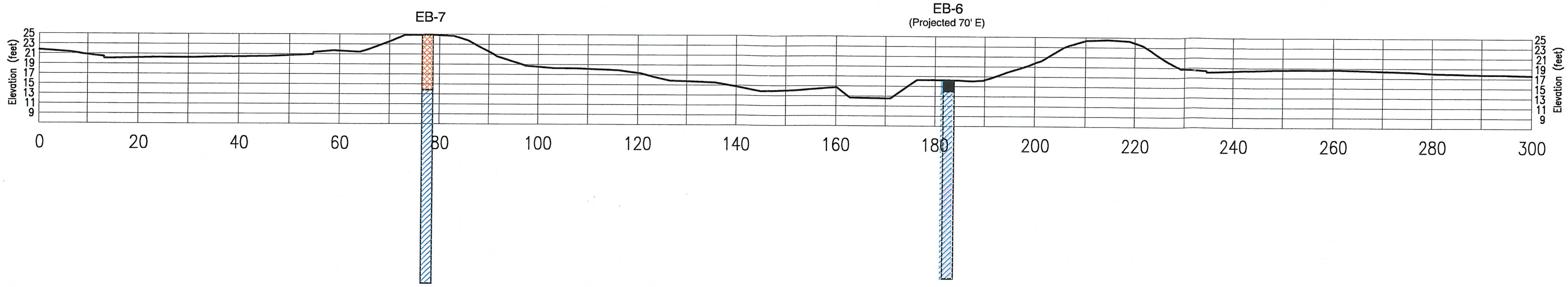
SECTION A AND B
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

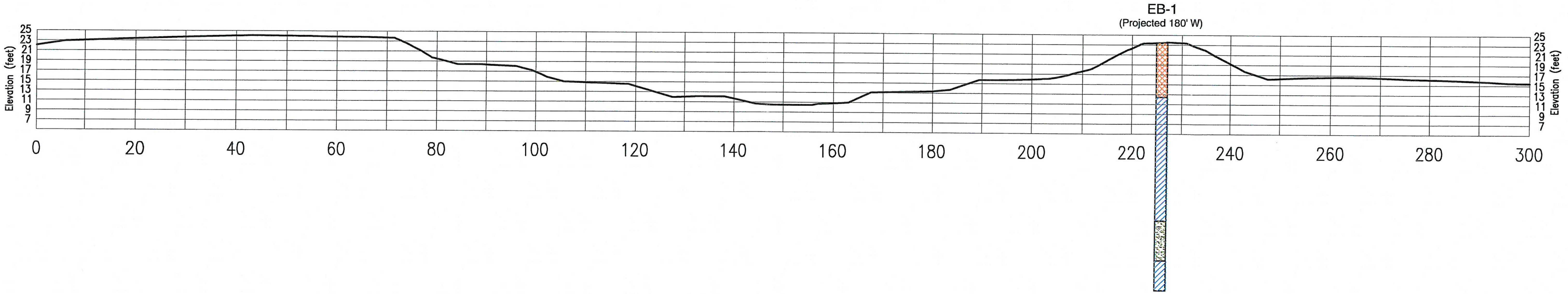
FIGURE 15A
1697-1

6/01*EB

SECTION C



SECTION D



LEGEND

▨	Levee Fill	Mostly very high plasticity clays, medium stiff to stiff, dark gray, dark brown and gray, moist
■	Native	
▨	Clay-Silt Found. Soils	Mostly high plasticity silty clays with sand, medium stiff to stiff, light brown to brown, moist to wet
▒	Sand-Silt Found. Soils	Mostly sands and silty sands, medium dense, gray, wet

Notes:
1.) Soil profile interpreted and generalized form boring data; conditions may be different away from the boring locations and between sample depths at boring locations.
2.) Refer to boring logs for more detailed information.
3.) Cross section locations shown on Figures 12A to 12F; cross sections are looking downstream

0 20
Scale feet

SECTION C AND D
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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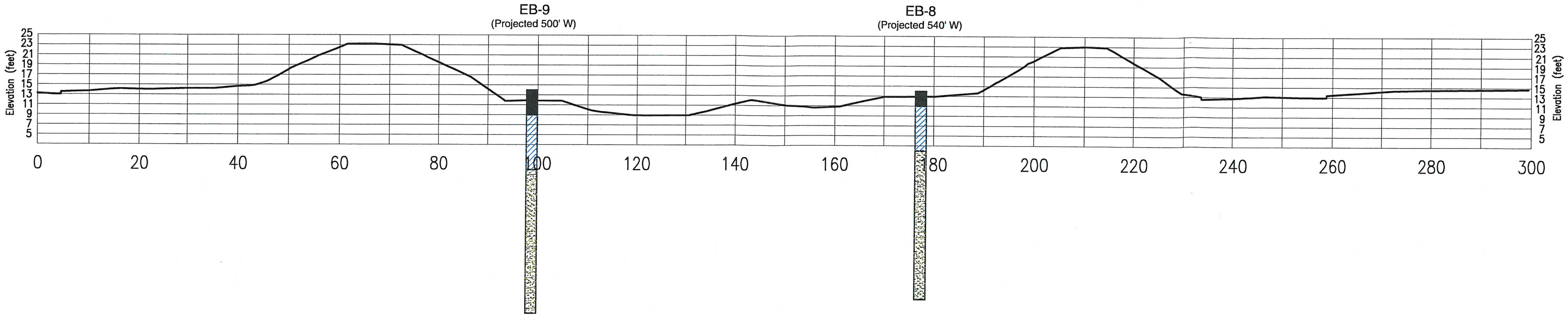
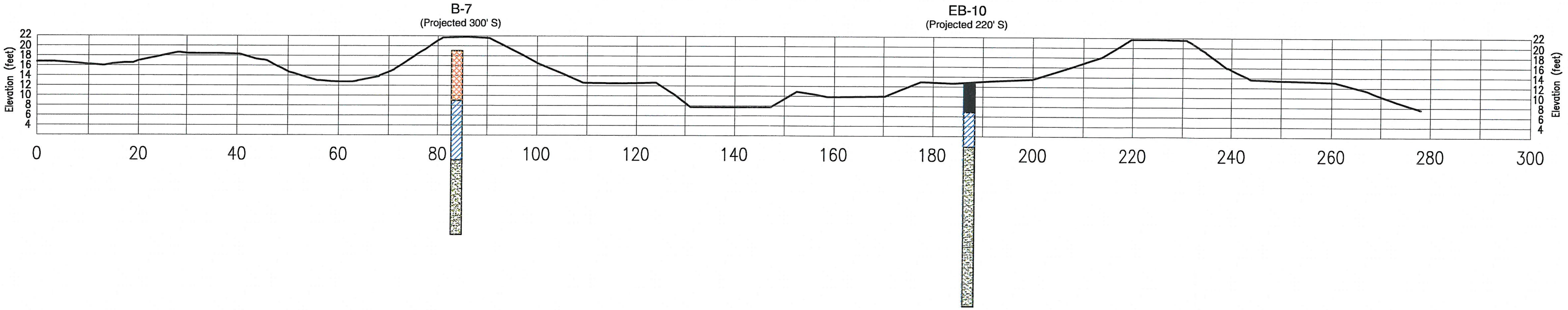
FIGURE 15B
1697-1

6/01*EB

SECTION E



SECTION F



LEGEND

▨ Levee Fill ■ Native	Mostly very high plasticity clays, medium stiff to stiff, dark gray, dark brown and gray, moist
▨ Clay-Silt Found. Soils	Mostly high plasticity silty clays with sand, medium stiff to stiff, light brown to brown, moist to wet
▒ Sand-Silt Found. Soils	Mostly sands and silty sands, medium dense, gray, wet

Notes:
1.) Soil profile interpreted and generalized form boring data; conditions may be different away from the boring locations and between sample depths at boring locations.
2.) Refer to boring logs for more detailed information.
3.) Cross section locations shown on Figures 12A to 12F; cross sections are looking downstream

0 20
Scale feet

SECTION E AND F
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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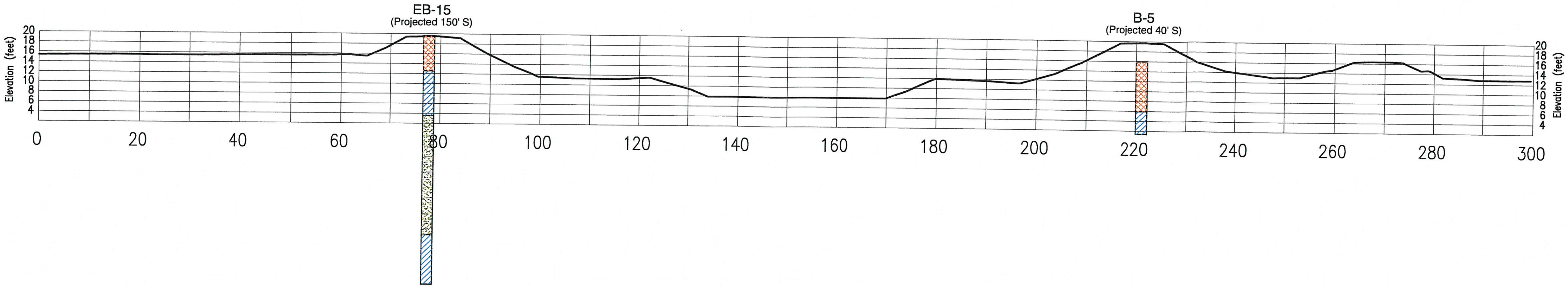
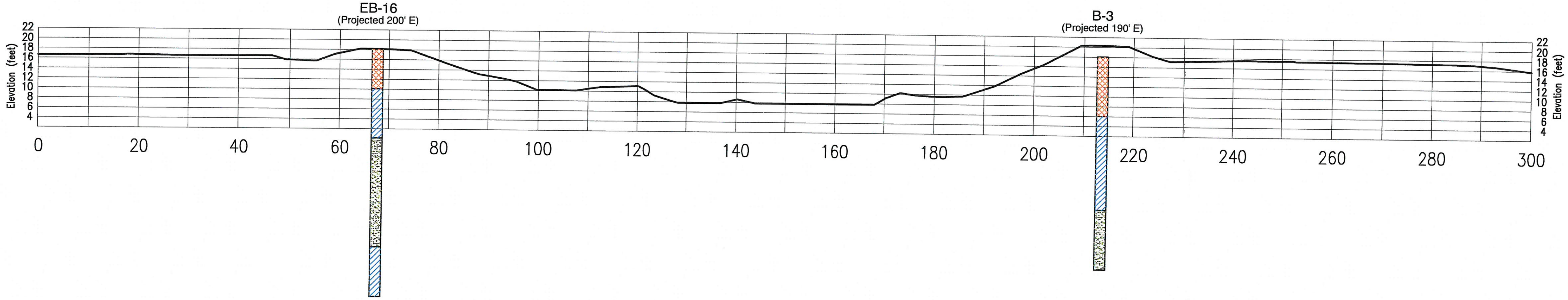
FIGURE 15C
1697-1

6/01*EB

SECTION G



SECTION H



LEGEND

▨ Levee Fill — Mostly very high plasticity clays, medium stiff to stiff, dark gray, dark brown and gray, moist

■ Native — Mostly very high plasticity clays, medium stiff to stiff, dark gray, dark brown and gray, moist

▨ Clay-Silt Found. Soils — Mostly high plasticity silty clays with sand, medium stiff to stiff, light brown to brown, moist to wet

▒ Sand-Silt Found. Soils — Mostly sands and silty sands, medium dense, gray, wet

Notes:

1.) Soil profile interpreted and generalized form boring data; conditions may be different away from the boring locations and between sample depths at boring locations.

2.) Refer to boring logs for more detailed information.

3.) Cross section locations shown on Figures 12A to 12F; cross sections are looking downstream

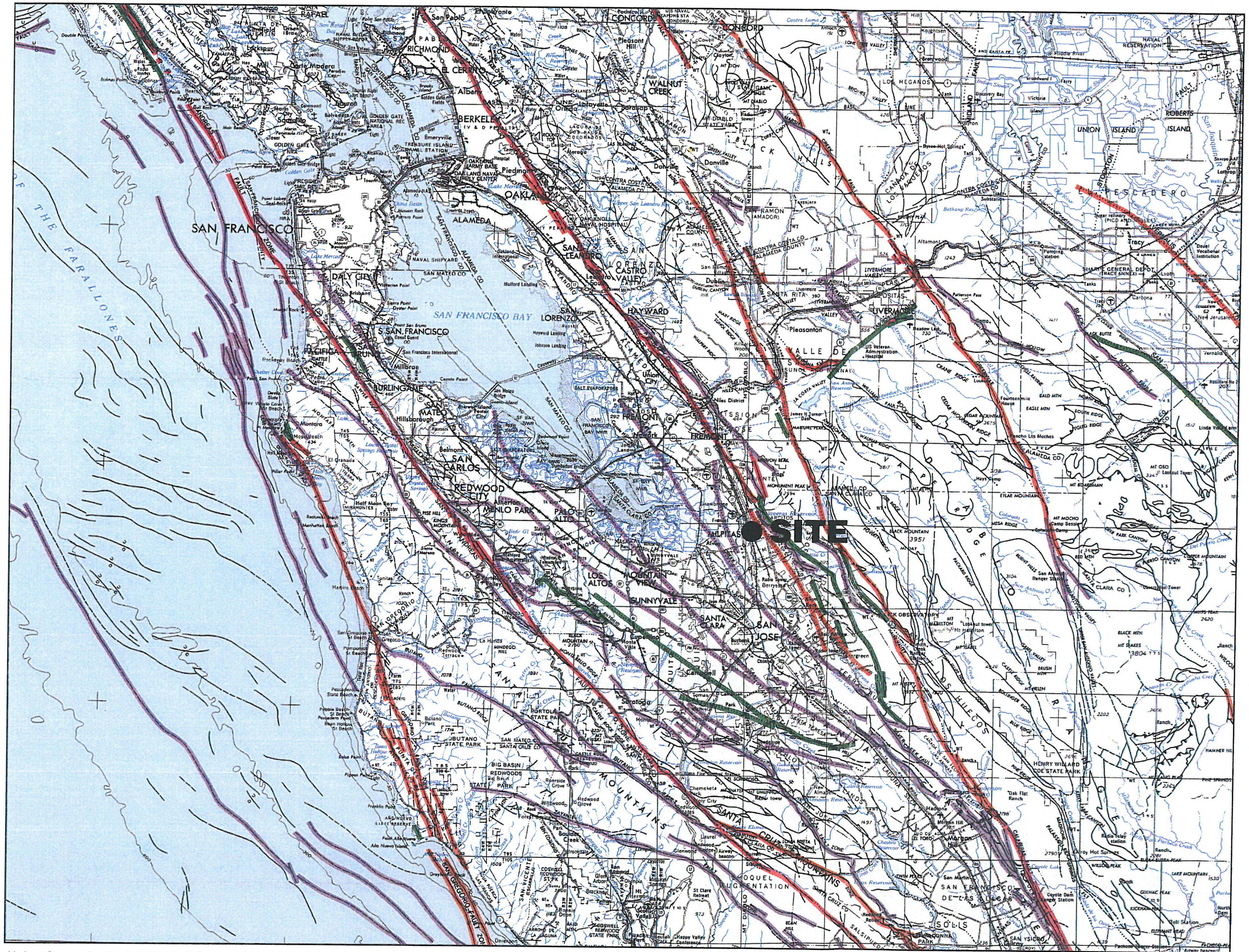
0 20
Scale feet

SECTION G AND H
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 15D
1697-1

6/01*EB



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Division of Mines and Geology, James F. Davis, State Geologist

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000	🟧		Displacement during Holocene time.[2]
		Pleistocene	700,000	🟩		Faults showing evidence of displacement during late Quaternary time.[3, 4]
	Early Quaternary		2,000,000	🟪		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		5,000,000	〜		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
	Miocene					

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122–A1–TF–250–00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120–A1–TF–250–00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000–meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

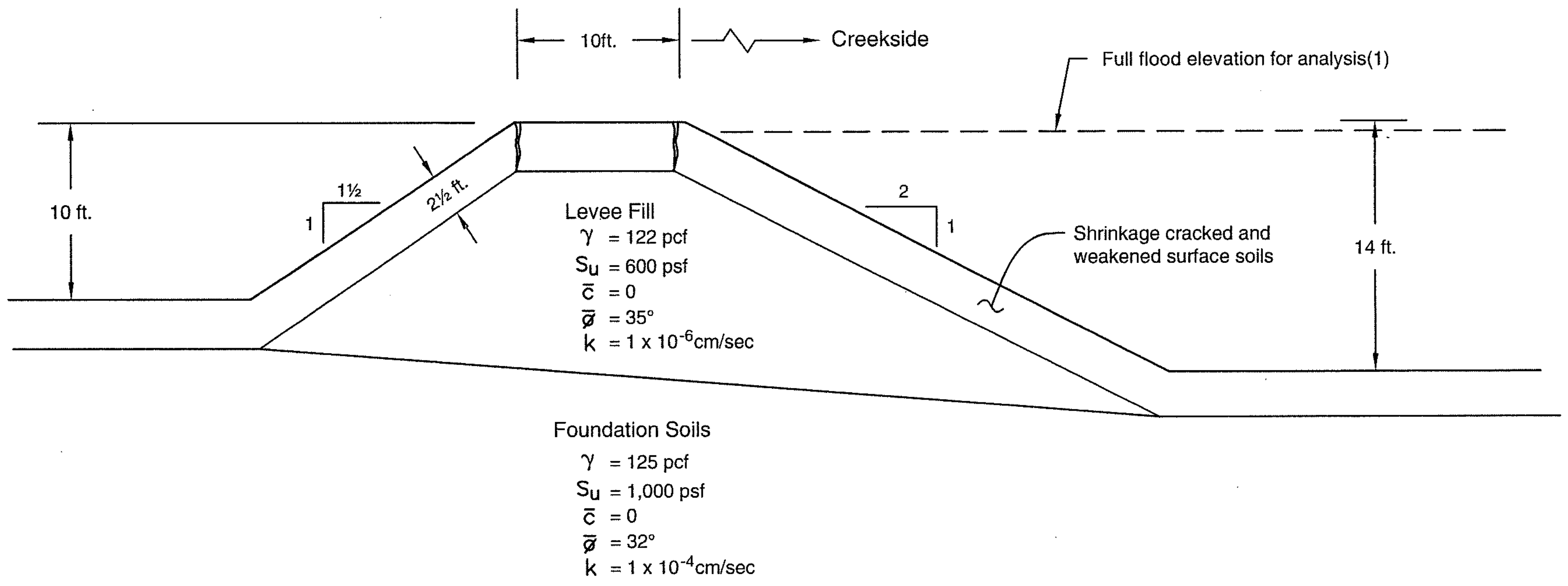
REGIONAL FAULT MAP
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 16

1697-1

6/01*EB



LEGEND

γ - Total density
S_u - Undrained shear strength
$\bar{c}$ - Effective cohesion
$\bar{\phi}$ - Effective friction angle
k - Hydraulic conductivity

Notes:

(1) Design flood elevation should be 3 ft lower to provide minimum required freeboard

Not To Scale

6/01*EB

SEEPAGE MODELING AND STABILITY ANALYSIS CROSS SECTION

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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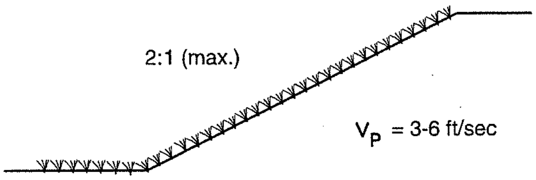
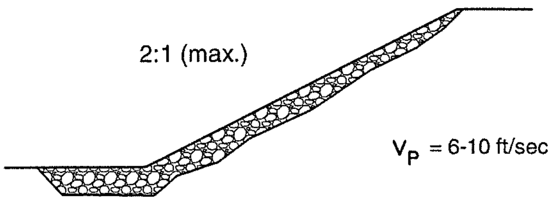
Environmental/Geotechnical/Engineering Services

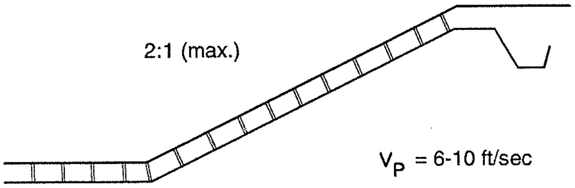
FIGURE 17
1697-1



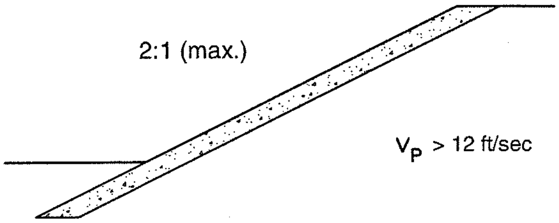
Grassed Soil Slope



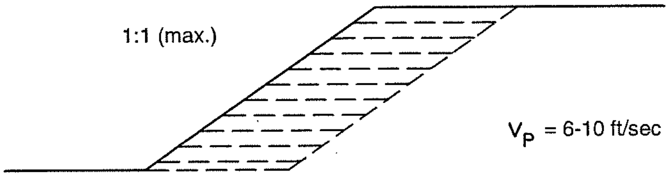
Riprap



Articulated Block Mat



Concrete Lining



Lime or Cement Treatment

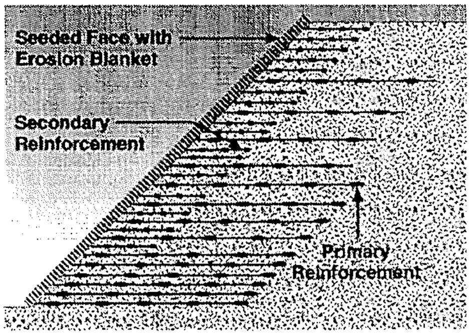
7/01*EB

SLOPE FACING ALTERNATIVES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

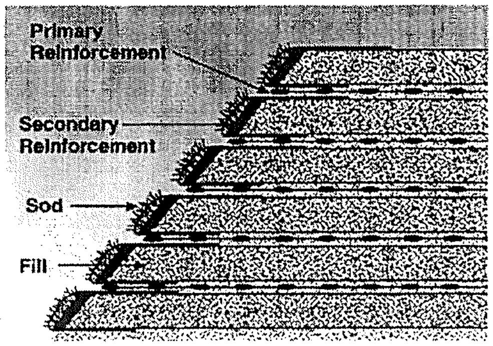
LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 18
1697-1



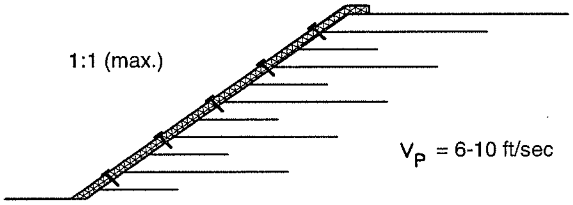
1:1 (max.) V_P = 6-10 ft/sec

Reinforced Slope

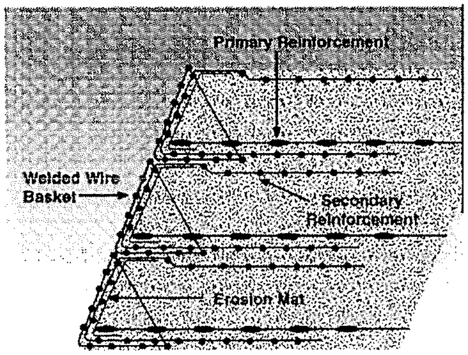


1:1 (max.) V_P = 6-10 ft/sec

Geosynthetic-Wrapped Face

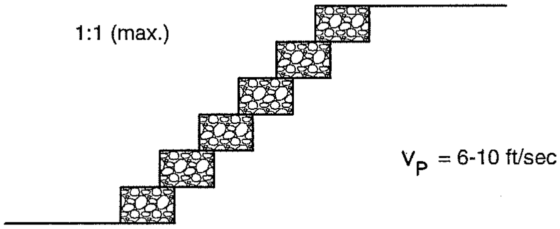


Erosion Control Mat Face



0.8:1 (max.) V_P = 6-10 ft/sec

Welded-Wire Baskets



Gabions

7/01*EB

REINFORCED SOIL SLOPE FACE ALTERNATIVES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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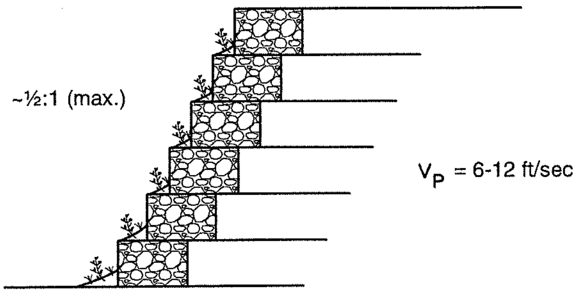
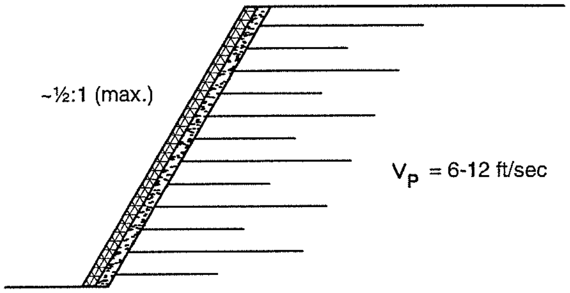
Environmental/Geotechnical/Engineering Services

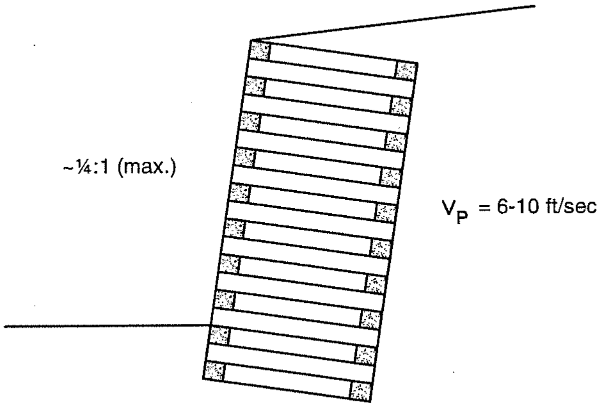
FIGURE 19
1697-1



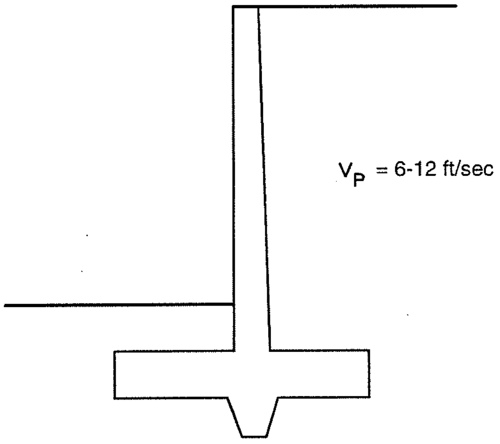
Gabions



Mechanically-Stabilized Earth Wall



Crib Wall



Reinforced Concrete Wall

7/01*EB

STRUCTURAL WALL ALTERNATIVES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 20
1697-1

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Environmental/ Geotechnical/ Engineering Services

8-28-01

Preliminary Geotechnical Report

Appendices

Berryessa Creek Levees Project
Santa Clara Valley Water District
Milpitas, California

Report No. 1697-1 has been prepared for:

Winzler & Kelly
495 Tesconi Circle, Santa Rosa, California 95401

August 28, 2001

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, rotary-wash auger drilling equipment. Seventeen 4 7/8-inch-diameter exploratory borings were drilled between April 16 and May 3, 2001, to a maximum depth of 50 feet. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings were approximately determined by portable Global Positioning System (GPS) equipment. Elevations of the borings were determined by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler that were hydraulically pushed. Adjacent to nine boring locations 6-inch Shelby Tube samplers were hydraulically pushed to obtain bulk samples. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Soil cuttings generated from the exploratory borings were placed in thirty-seven 55-gallon drums, labeled, and temporarily stored on-site pending environmental analysis. Five random samples from the drums were collected, as per the requirements of our drum disposal subcontractor, Clearwater Environmental Management, Inc. (Clearwater). These five samples were sent to Sequoia Analytical, a California State Certified Laboratory, where the samples were composited into one sample and tested for Lead, Total Purgeable Hydrocarbons (C6-C12), Benzene, Toluene, Ethylbenzene, Xylenes, and Methyl tert-butyl ether. The results of the environmental analysis were forwarded to Clearwater, who approved the drums for disposal as a non-hazardous waste. On May 24th, 2001, we met with Clearwater at the site for drum removal. Thirty-seven drums were removed from the site by Clearwater and taken to Alviso Independent Oil in Alviso, California for disposal. The Non-Hazardous Waste Manifest from Clearwater Environmental Management, Inc. is attached following the boring logs.

The attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * * *

PRIMARY DIVISIONS				SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, gravel—sand mixtures, little or no fines
				GP	○	Poorly graded gravels or gravel—sand mixtures, little or no fines
			GRAVEL WITH FINES	GM	○	Silty gravels, gravel—sand—silt mixtures, plastic fines
				GC	▨	Clayey gravels, gravel—sand—clay mixtures, plastic fines
		SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	⁙	Well graded sands, gravelly sands, little or no fines
				SP	∴	Poorly graded sands or gravelly sands, little or no fines
			SANDS WITH FINES	SM	⁙	Silty sands, sand—silt—mixtures, non—plastic fines
				SC	▨	Clayey sands, sand—clay mixtures, plastic fines
FINE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML	▥	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
				CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	☰	Organic silts and organic silty clays of low plasticity
		SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
				CH	▨	Inorganic clays of high plasticity, fat clays
				OH	≋	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	⁂	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

	200		40		10		4		3/4"		3"		12"	

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 — 0.4 — 2 — 5 — 19 — 76mm

GRAIN SIZES

☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION

⧗ MODIFIED CALIFORNIA

▯ D&M UNDERWATER SAMPLER

■ SHELBY TUBE

○ NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0—4
LOOSE	4—10
MEDIUM DENSE	10—30
DENSE	30—50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0—1/4	0—2
SOFT	1/4—1/2	2—4
MEDIUM STIFF	1/2—1	4—8
STIFF	1—2	8—16
VERY STIFF	2—4	16—32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2—inch O.D. (1—3/8 inch I.D.) split spoon (ASTM D—1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D—1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D—2487)

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A—1

EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-20-01 FINISH DATE: 4-20-01

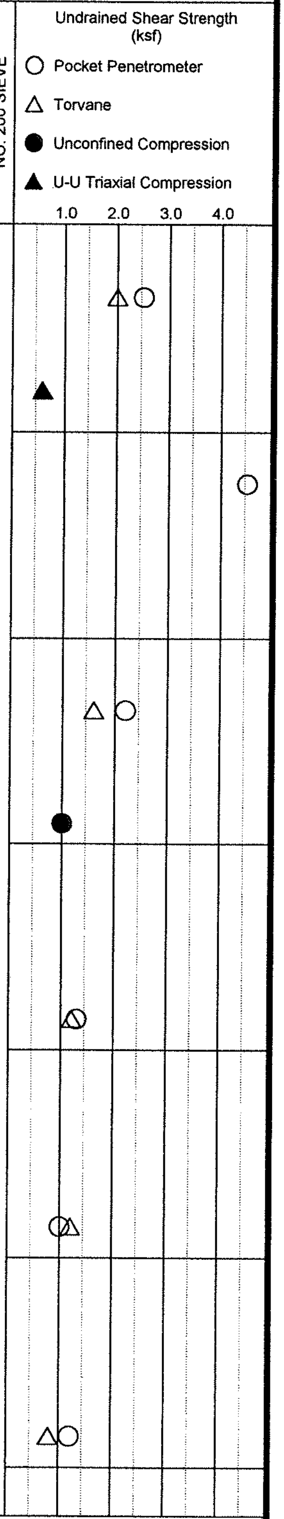
PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 24 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
24	0						
23.5		6 inches aggregate base	AB				
		SANDY CLAY (CL) [FILL] stiff, moist, brown, fine to coarse sand, some fine to coarse gravel, low plasticity	CL	22	23	101	
21.8		**SILTY CLAY (CH) [FILL]** very stiff to hard, moist, dark gray-brown to black, high plasticity	CH				
	5	(CU Triaxial Test)					
				50	19		
		Plasticity Index = 54, Liquid Limit = 83 (Moisture/Density-Porosity Test, Consolidation Test, Hydraulic Conductivity Test)					
	10						
13.5		**SILTY CLAY (CH)** stiff, moist, brown to light brown, some fine to medium sand, high plasticity		22	29	92	
	15	Plasticity Index = 60, Liquid Limit = 82 (CU-PP Triaxial Test)					
			CH	23	25	99	
	20						
				12	32	91	
	25	wet, trace fine gravel					
-4.0		**SANDY SILT (ML)** medium stiff, wet, olive, fine, trace clay, low plasticity	ML	11	26	98	
-6.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,895
Easting: 6,155,150

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-20-01 FINISH DATE: 4-20-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-6	30		ML					
-7.0		**SILTY SAND (SM)** medium dense, wet, olive, fine	SM	18	☒			
	35							
-12.5		**SILTY GRAVEL (GM)** dense, wet, gray-brown, fine to coarse, some fine to coarse sand	GM	56	☒			
	40	loss of circulation		22	☒			
-20.5	45	**SANDY CLAY (CL)** stiff, wet, brown fine to medium, some fine gravel, low plasticity	CL	38	⧗			
-26.0	50	Bottom of Boring at 50 feet						
	55							
	60							

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,984,895
Easting:	6,155,150

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-27-01 FINISH DATE: 4-27-01

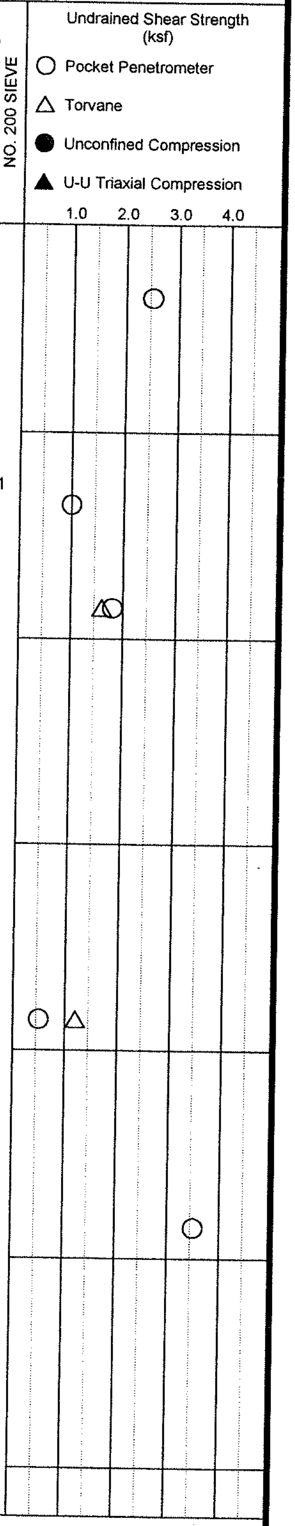
PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 20 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
20	0	**SANDY CLAY (CL)** stiff to very stiff, damp, light brown, fine sand, moderate plasticity Plasticity Index = 19, Liquid Limit = 36	CL	20	19	109	
		trace fine gravel					
	5	Plasticity Index = 10, Liquid Limit = 28 (Consolidation Test, Moisture/Density-Porosity Test, Hydraulic Conductivity Test)		16	24 21	106	61
11.3	10	**SILTY CLAY (CL)** stiff, wet, light brown to olive-brown, some fine sand, low plasticity	CL	18	22	106	
		increasing fine to coarse sand, some fine gravel					
6.5	15	**SANDY CLAY (CL)** medium stiff to stiff, wet, light brown, fine to medium, low plasticity	CL	12			
	20			28	19	111	
-1.0	25	**SILTY CLAY (CL)** very stiff, wet, light brown, trace fine gravel, low plasticity	CL	39			
-7.5		**CLAYEY SAND (SC)** medium dense, wet, brown, fine	SC	26	20	108	
-10.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,588
Easting: 6,157,317

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-27-01 FINISH DATE: 4-27-01	COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-10	30	▨		SC					
-10.5		▨	**SILTY CLAY (CL)** very stiff, wet, brown, some fine sand, low plasticity	CL	40	⧗			
	35	▨							
-16.5		▨	**SANDY CLAY (CL)** very stiff, wet, brown, fine sand, low plasticity	CL	40	⧗	16	114	
-20.0	40		Bottom of Boring at 40 feet						
	45								
	50								
	55								
	60								

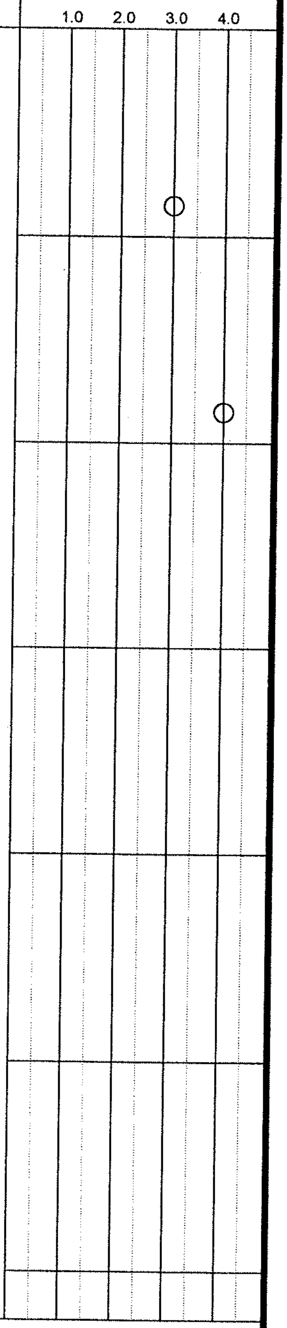
Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression



GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,588

Easting: 6,157,317

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-18-01 FINISH DATE: 4-18-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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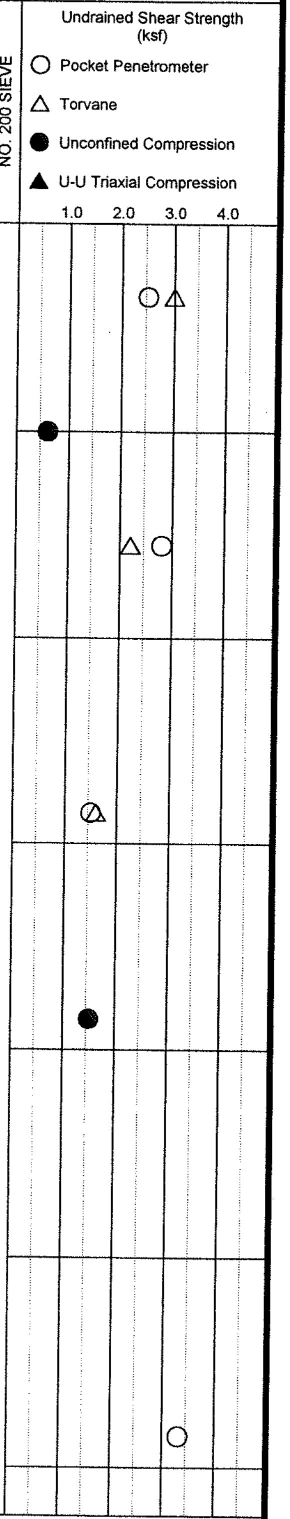
MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 31 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
31	0						
30.5		6 inches aggregate base	AB				
		SILTY CLAY (CH) [FILL] very stiff, moist, dark gray-brown to black, some fine sand, trace fine gravel, high plasticity		24	22	101	
	5	Plasticity Index = 45, Liquid Limit = 71	CH				
				25	19	102	
21.3	10	**SANDY CLAY (CL)** stiff, moist, gray to olive-brown, fine to coarse sand, moderate plasticity Plasticity Index = 30, Liquid Limit = 49 (Moisture/Density-Porosity Test, Consolidation Test, Hydraulic Conductivity Test)					
			CL	13	23	101	
	15						
		Plasticity Index = 14, Liquid Limit = 29 (CU-PP Triaxial Test, Hydraulic Conductivity Test)					
12.0		**CLAYEY SAND (SC)** medium dense, moist, light brown, fine to medium		14			
	20		SC				
		wet, color to olive		19	17		
	25						
4.0		**SANDY CLAY (CL)** very stiff, wet, light brown, fine sand, low plasticity	CL				
				19			
1.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,894
Easting: 6,157,097

LA CORP.GDT 7/23/01 MV*FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-18-01 FINISH DATE: 4-18-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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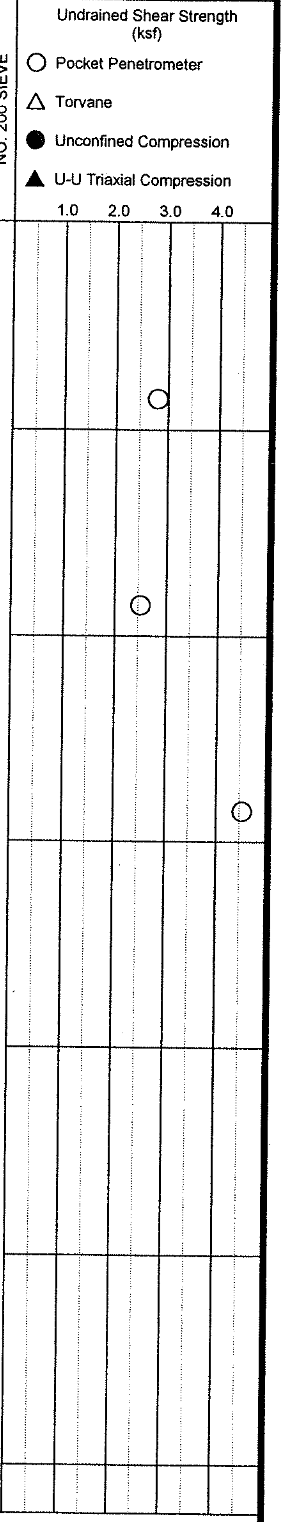
ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
1	30	▨	**SANDY CLAY (CL)** very stiff, wet, light brown, fine sand, low plasticity	CL					
-2.0		▨	**CLAYEY SAND (SC)** medium dense, wet, light brown, fine to medium	SC					
-3.0		▨	**SANDY CLAY (CL)** very stiff, wet, light brown, fine to coarse sand, trace coarse gravel, low plasticity	CL	22	⧗			
	35	▨							
	40	▨			37	⧗	16	116	
		▨	color to brown, trace fine gravel		58	⧗			
-14.0	45		Bottom of Boring at 45 feet						
	50								
	55								
	60								



LA CORP.GDT 7/23/01 MV* FLL

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,894
Easting: 6,157,097

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-27-01 FINISH DATE: 4-27-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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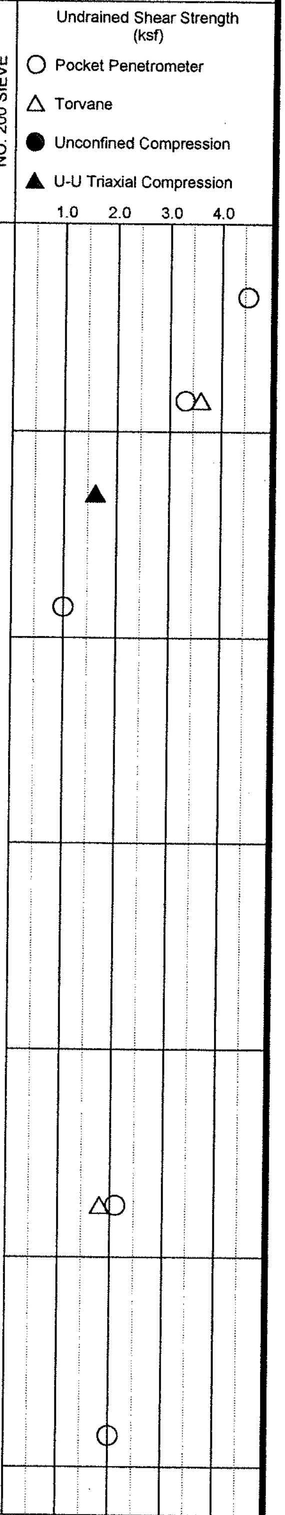
MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 27 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
27	0	**SILTY CLAY (CL) [FILL]** very stiff, moist, brown, fine to coarse sand, fine to coarse gravel, moderate plasticity (Particle Size Test, R-value) Plasticity Index = 17, Liquid Limit =34	CL	40	10	122	60
24.0		**SILTY CLAY (CH) [FILL]** very stiff, moist, dark gray to black, high plasticity (CU Triaxial Test)	CH	30	27	94	
19.0		**SANDY CLAY (CL)** medium stiff to stiff, wet, light brown to olive-brown, fine to medium sand, low plasticity	CL	11	21	102	
17.0	10	**CLAYEY SAND (SC)** medium dense, wet, light brown, fine to medium, some coarse, some fine gravel	SC	13			
7.0	20	**SILTY CLAY (CL)** stiff, wet, brown, some fine sand, low plasticity	CL	32	18	116	
-0.5		**CLAYEY SAND (SC)** medium dense, wet, brown, fine to coarse, some fine gravel	SC	33	18	113	
-2.5			CL				
-3.0	30						



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GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,172
Easting: 6,156,962

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-27-01 FINISH DATE: 4-27-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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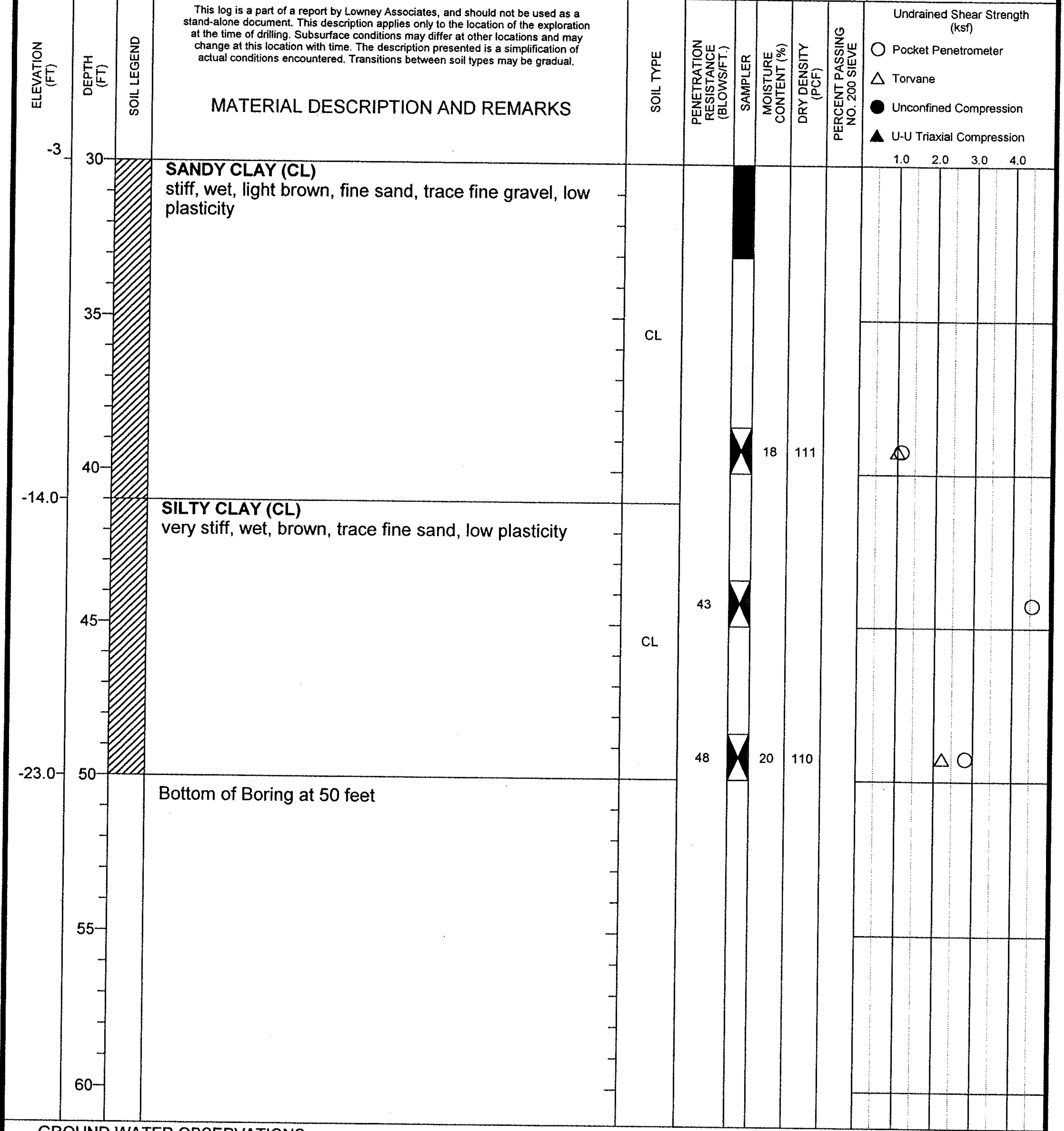

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-3	30	**SANDY CLAY (CL)** stiff, wet, light brown, fine sand, trace fine gravel, low plasticity	CL				
	40				18	111	
-14.0		**SILTY CLAY (CL)** very stiff, wet, brown, trace fine sand, low plasticity	CL				
	45			43			
				48	20	110	
-23.0	50	Bottom of Boring at 50 feet					
	55						
	60						

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,984,172
Easting:	6,156,962

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-5

Sheet 1 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-16-01 FINISH DATE: 4-16-01	COMPLETION DEPTH: 40.5 FT.

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MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 19 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
19	0	**SANDY CLAY (CL)** very stiff, damp, light brown, fine to coarse sand, low plasticity	CL				
				19	17	105	
		moist		35	23	100	
	5						
				30	19	110	
	10						
		trace fine gravel					
5.3		**CLAYEY SAND (SC)** medium dense, wet, light brown, fine to coarse	SC				
	15			25	17	113	
2.0		**SANDY CLAY (CL)** stiff to very stiff, wet, light brown, fine to coarse sand, some fine gravel, low plasticity	CL				
	20			38	16	115	
	25			23	19	109	
-11.0	30			33			

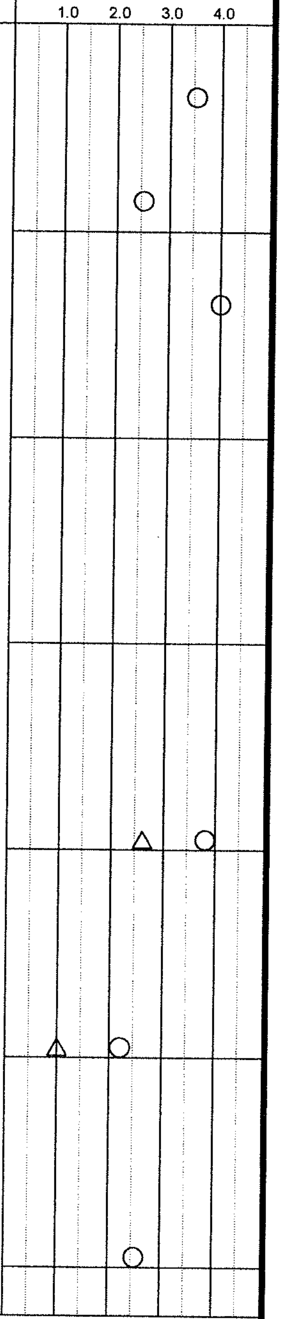
Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression



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GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,158

Easting: 6,156,781

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-5 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-16-01 FINISH DATE: 4-16-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 40.5 FT.

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Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
-11	30	▨	**SANDY CLAY (CL)** stiff to very stiff, wet, light brown, fine to coarse sand, some fine gravel, low plasticity	CL		⧗				
-14.0	35	▨	**CLAYEY SAND (SC)** medium dense, wet, light brown, fine to coarse	SC	47	⧗				
-18.5		▨	**SANDY CLAY (CL)** stiff, wet, light brown, fine to coarse sand, low plasticity	CL						
-21.0	40				40	⧗	17	113		○
			Bottom of Boring at 40 feet							
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,158
Easting: 6,156,781

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-6

Sheet 1 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-17-01 FINISH DATE: 4-17-01	COMPLETION DEPTH: 40.0 FT.

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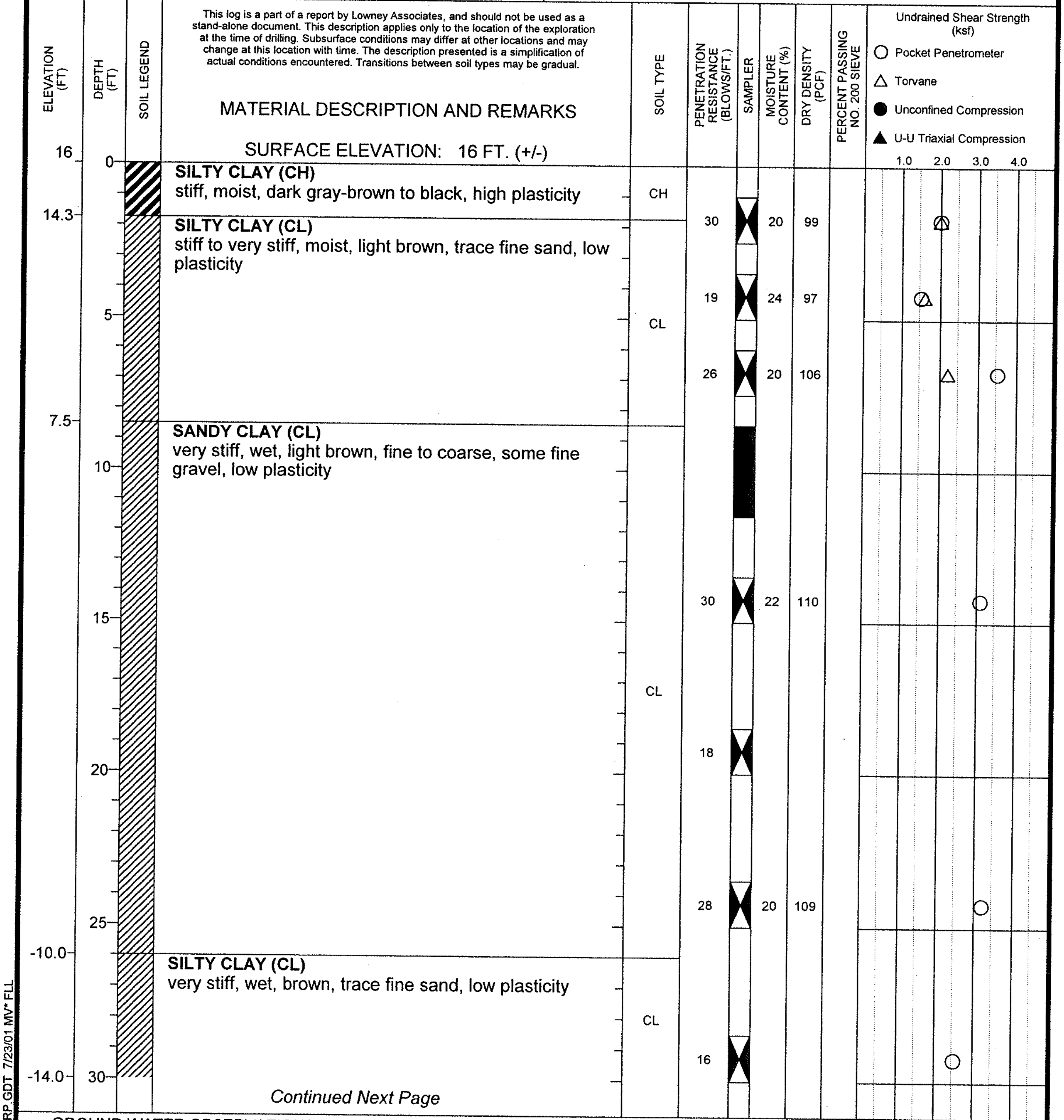

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 16 FT. (+/-)

ELEVATION (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE
16	**SILTY CLAY (CH)** stiff, moist, dark gray-brown to black, high plasticity	CH
14.3	**SILTY CLAY (CL)** stiff to very stiff, moist, light brown, trace fine sand, low plasticity	CL
7.5	**SANDY CLAY (CL)** very stiff, wet, light brown, fine to coarse, some fine gravel, low plasticity	CL
-10.0	**SILTY CLAY (CL)** very stiff, wet, brown, trace fine sand, low plasticity	CL
-14.0	*Continued Next Page*	

PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
30	20	99
19	24	97
26	20	106
30	22	110
18		
28	20	109
16		

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,575

Easting: 6,156,220

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-6 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-17-01 FINISH DATE: 4-17-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 40.0 FT.

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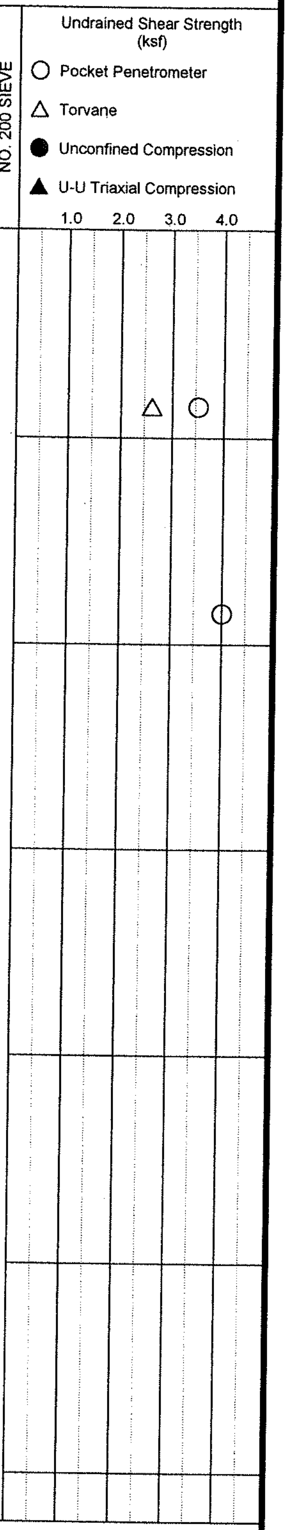
ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-14	30	▨	**SILTY CLAY (CL)** very stiff, wet, brown, trace fine sand, low plasticity	CL					
	35	▨			49	⧗	16	116	
		▨			46	⧗			
-24.0	40		Bottom of Boring at 40 feet						
	45								
	50								
	55								
	60								



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,575
Easting: 6,156,220

EXPLORATORY BORING: EB-7

Sheet 1 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-19-01 FINISH DATE: 4-19-01	COMPLETION DEPTH: 50.0 FT.

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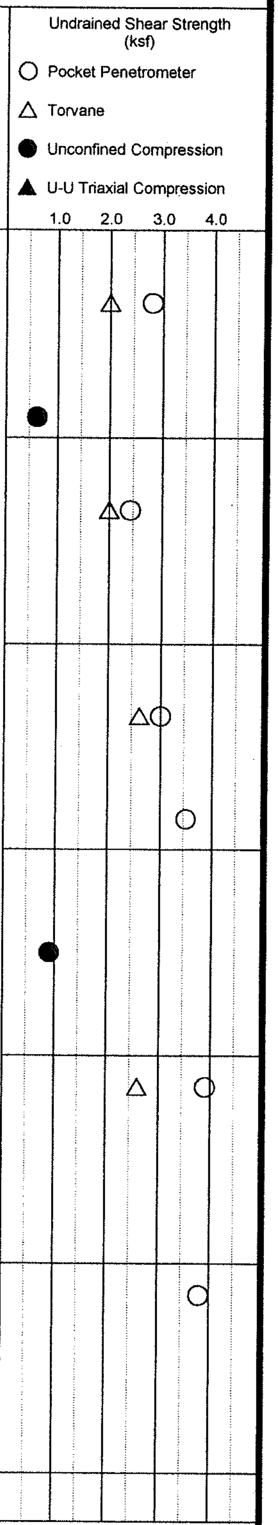
ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
25	0	SURFACE ELEVATION: 25 FT. (+/-)					
24.5		6 inches aggregate base	AB				
		SILTY CLAY (CH) [FILL] very stiff, moist, dark gray-brown to black, some fine sand, trace fine gravel, high plasticity	CH	23	22	100	
22.5		**SILTY CLAY (CH) [FILL]** very stiff, moist, dark gray-brown to black, some fine sand, trace fine gravel, high plasticity Plasticity Index = 51, Liquid Limit = 72 (Particle Size Test, CU-PP Triaxial Test)	CH				92
	5	Plasticity Index = 53, Liquid Limit = 75		15	31	83	
15.5	10	**SILTY CLAY (CL)** very stiff, moist, light gray to olive-brown, some fine sand, low plasticity	CL	28	22	106	
				25			
	15	(Particle Size Test, CU-PP Triaxial Test, Consolidation Test, Moisture/Density-Porosity Test) Plasticity Index = 20, Liquid Limit = 34					67
5.0	20	**SANDY CLAY (CL)** very stiff to hard, wet, light brown, fine sand, trace coarse sand, trace fine gravel	CL	33	17	113	
	25		CL	30			
-5.0	30						



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GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,470
Easting: 6,156,204

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-7 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-19-01 FINISH DATE: 4-19-01	COMPLETION DEPTH: 50.0 FT.

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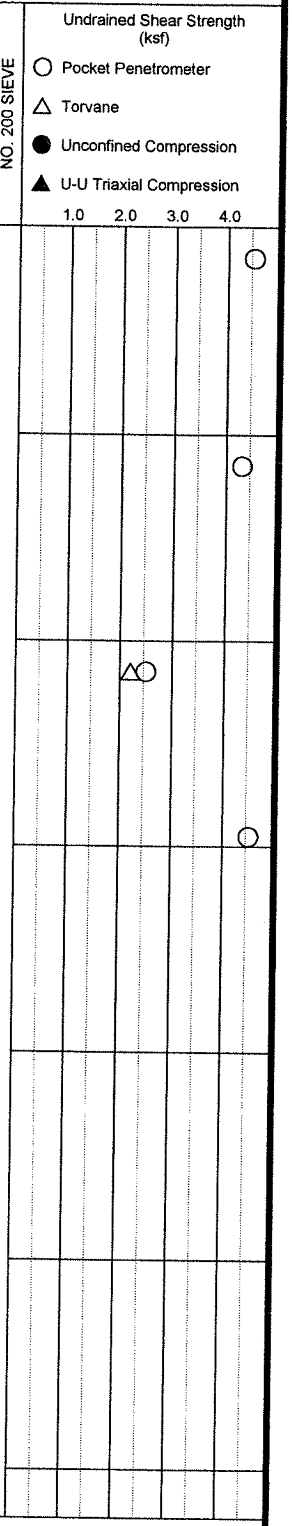
ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-5	30	▨	**SANDY CLAY (CL)** very stiff to hard, wet, light brown, fine sand, trace coarse sand, trace fine gravel color grades to brown		52	⧗	17	114	
	35	▨			40	⧗			
	40	▨		CL	30	⧗	17	113	
	45	▨			60	⧗			
		▨			60	⧗			
-25.0	50		Bottom of Boring at 50 feet						
	55								
	60								



GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,984,470

Easting: 6,156,204

EXPLORATORY BORING: EB-8

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-17-01 FINISH DATE: 4-17-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 41.5 FT.

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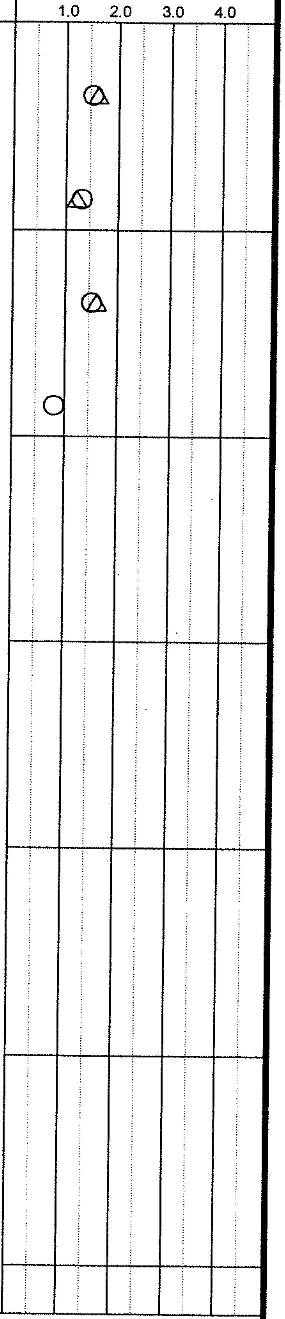
ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION: 14 FT. (+/-)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
14	0	**SILTY CLAY (CH)** stiff, moist, dark gray-brown to black, some fine sand, high plasticity	CH	10	32	87	
11.3		**SILTY CLAY (CL)** stiff, moist, light brown, some fine sand, low plasticity	CL	13	31	90	
	5			14	21	105	
5.8		**SANDY SILT (ML)** medium stiff, wet, light brown, fine sand, low plasticity	ML	6	23	103	
3.5	10	**SILTY SAND (SM)** medium dense, wet, gray, fine to medium, trace coarse, some clay	SM				
	15	(Particle Size Test)		17			40
	20			12	24		
	25			17	16		
-15.0			CL				
-16.0	30	*Continued Next Page*					

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,097
Easting: 6,156,204

LA_CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-8 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-17-01 FINISH DATE: 4-17-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 41.5 FT.

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Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
-16	30	▨	**SANDY CLAY (CL)** stiff, wet, brown, fine to medium sand, low plasticity	CL	17	☒				
-20.0	35	▥	**CLAYEY SILT (ML)** stiff, wet, blue-gray, with fine to medium sand (Particle Size Test)	ML	17	■			73	
	40	▥			23	■				
-27.5			Bottom of Boring at 41½ feet							
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,097
Easting: 6,156,204

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-16-01 FINISH DATE: 4-16-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 45.0 FT.

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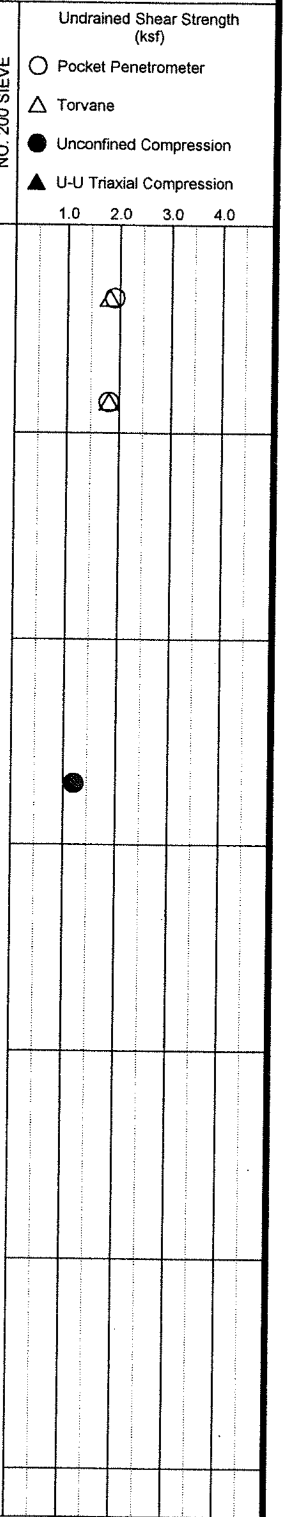
MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 14 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
14	0	**SILTY CLAY (CH)** stiff, moist, dark gray-brown to black, some fine sand, high plasticity Plasticity Index = 59, Liquid Limit = 85	CH	17	30	86	
9.5				18	30	92	
	5	**SILTY CLAY (CL)** medium stiff, moist, light brown, some fine sand, low plasticity	CL				
3.5	10	**CLAYEY SAND (SC)** loose, wet, light brown, fine to medium Plasticity Index = 2, Liquid Limit = 23 (CU-PP Triaxial Test, Consolidation Test, Moisture/Density-Porosity Test)	SC	7			
-2.0	15	**SAND WITH SILT (SP-SM)** medium dense, wet, gray-brown, fine to coarse, trace fine gravel	SP-SM				
	20			15	17		9
		medium and coarse, some fine, trace fine to coarse gravel					
	25			29	11		6
-16.0	30	dense		34	12		


Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,014
Easting: 6,154,223

LA_CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-9 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-16-01 FINISH DATE: 4-16-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 45.0 FT.

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Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-16 -16.0	30	**SAND WITH SILT (SP-SM)** dense, wet, gray-brown, medium and coarse, some fine, trace fine to coarse gravel	SP-SM					
		increased fine to coarse gravel		34	☒	12		
	35							
-22.0		**SANDY CLAY (CL)** stiff, wet, brown, trace fine gravel	CL					
-24.0		**SAND WITH SILT (SP-SM)** dense, wet, gray-brown, fine to coarse, trace fine gravel	SP-SM	32	☒	15		
	40							
-26.5		**SAND (SP)** dense, wet, gray-brown, medium and coarse, some fine, some fine to coarse gravel	SP	40	☒	15		
-31.0	45	Bottom of Boring at 45 feet						
	50							
	55							
	60							

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,014
Easting: 6,154,223

LA_CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-10

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-18-01 FINISH DATE: 4-18-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 45.0 FT.

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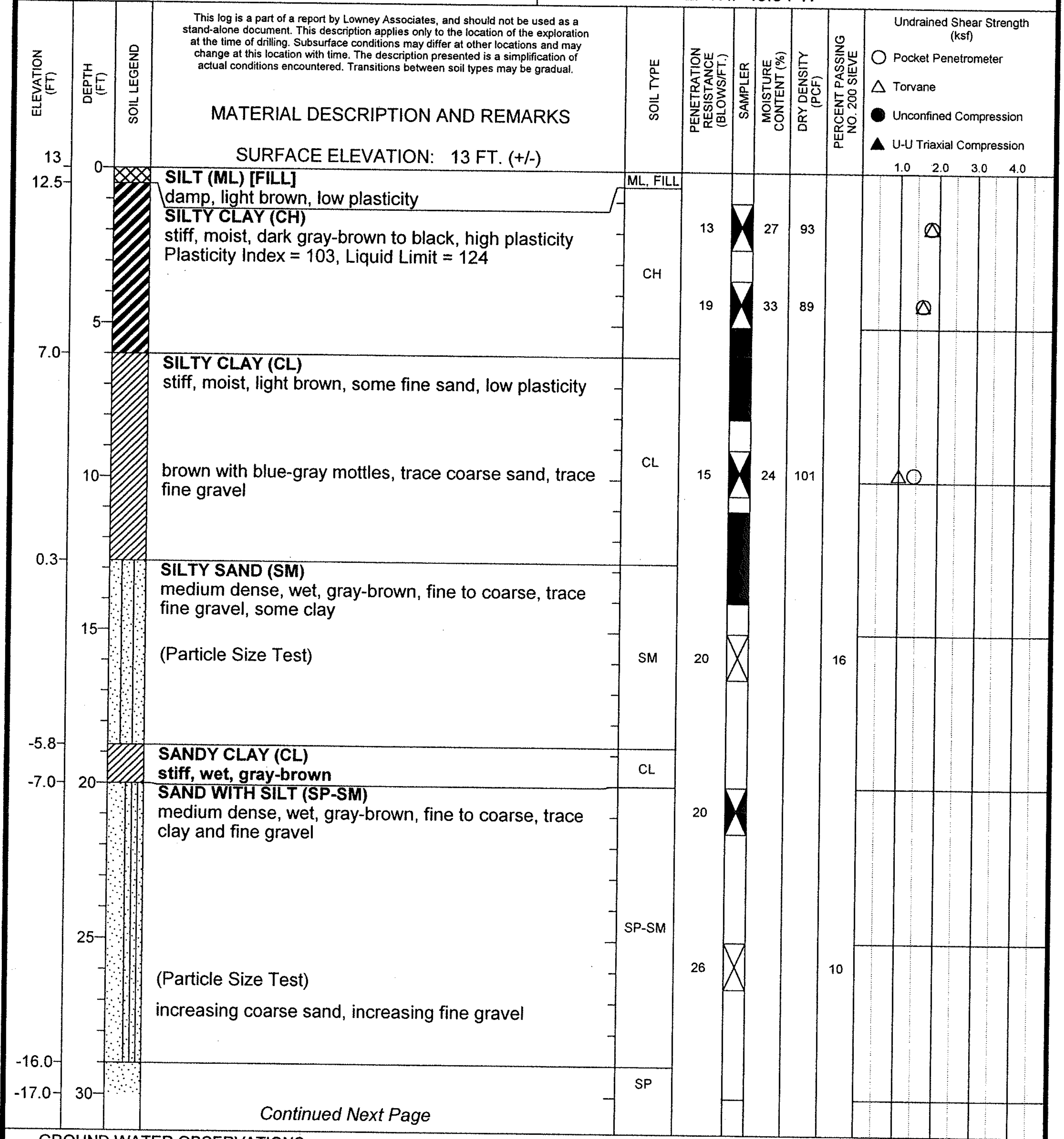

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 13 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
13	0	**SILT (ML) [FILL]** damp, light brown, low plasticity	ML, FILL				
12.5		**SILTY CLAY (CH)** stiff, moist, dark gray-brown to black, high plasticity Plasticity Index = 103, Liquid Limit = 124	CH	13	27	93	
				19	33	89	
	5						
7.0		**SILTY CLAY (CL)** stiff, moist, light brown, some fine sand, low plasticity	CL				
	10	brown with blue-gray mottles, trace coarse sand, trace fine gravel		15	24	101	
0.3		**SILTY SAND (SM)** medium dense, wet, gray-brown, fine to coarse, trace fine gravel, some clay	SM				
	15	(Particle Size Test)		20			16
-5.8		**SANDY CLAY (CL)** stiff, wet, gray-brown	CL				
-7.0	20	**SAND WITH SILT (SP-SM)** medium dense, wet, gray-brown, fine to coarse, trace clay and fine gravel	SP-SM	20			
	25	(Particle Size Test)		26			10
		increasing coarse sand, increasing fine gravel					
-16.0			SP				
-17.0	30						

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,838
Easting: 6,153,175

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-10 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-18-01 FINISH DATE: 4-18-01	COMPLETION DEPTH: 45.0 FT.

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Undrained Shear Strength (ksf)
- ○ Pocket Penetrometer
- △ Torvane
- ● Unconfined Compression
- ▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
-17	30		**GRAVELLY SAND (SP)** dense, wet, gray-brown, coarse, some fine to medium, fine to coarse gravel, trace clay (Particle Size Test)	SP	30	☒			6	
-21.0			**SANDY CLAY (CL)** stiff, wet, blue-gray, fine to medium	CL	20	⧗	20	109		△○
	35									
-25.0			**SAND WITH SILT (SP-SM)** medium dense to dense, wet, gray-brown, fine to coarse, trace fine gravel	SP-SM						
	40				19	☒				
					45	☒				
-32.0	45		Bottom of Boring at 45 feet							
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,838

Easting: 6,153,175

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-11

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 5-10-01 FINISH DATE: 5-10-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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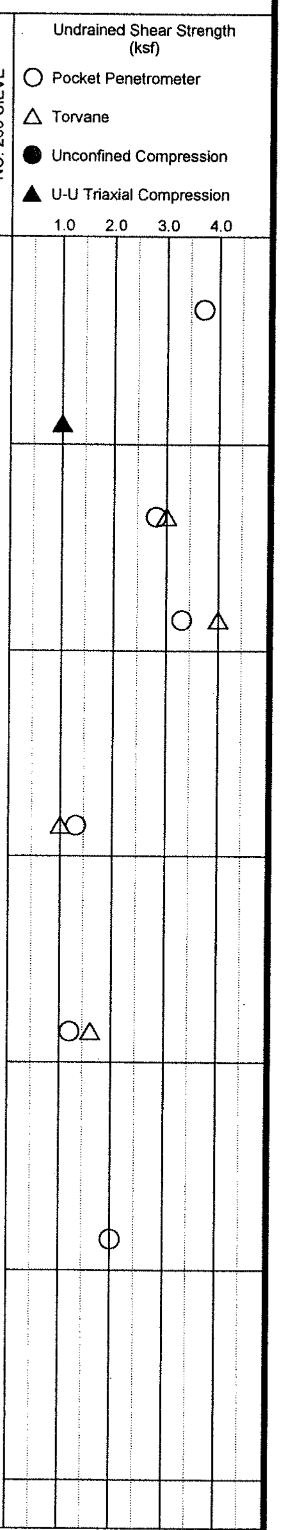
SURFACE ELEVATION: 22 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
22	0	▨	**SILTY CLAY (CL) [FILL]** stiff to very stiff, damp, brown to dark brown mottled, some fine to medium sand, trace fine gravel, low plasticity	CL	39	17	111	
	5	▨	(CU Triaxial Test, Hydraulic Conductivity Test)					
		▨	moist, black mottles		32	24	98	
14.3		▨	**SILTY CLAY (CH)** very stiff, wet, black, high plasticity	CH	33	25	100	
	10	▨						
9.0		▨	**SILTY CLAY (CL)** stiff, wet, dark olive-brown, trace fine sand, low plasticity	CL	9	32	90	
	15	▨						
	20	▨				23	105	
-0.5		▨	**SAND WITH SILT (SP-SM)** medium dense, wet, brown with gray mottles, fine to medium, some coarse	SP-SM	16	24	102	
	25	▨	trace fine gravel					
-8.0	30		*Continued Next Page*					



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,247
Easting: 6,153,484

LA CORP.GDT 7/23/01 MV* FLL

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Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-11 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 5-10-01 FINISH DATE: 5-10-01	COMPLETION DEPTH: 50.0 FT.

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ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression (1.0 2.0 3.0 4.0)
-8 -8.0	30		**SILTY SAND (SM)** medium dense, wet, brown with gray mottles, fine to medium, some coarse, some clay (Particle Size Test)	SM	21	⊠			13	
	35				38	⊠				
-16.0			**GRAVELLY SAND WITH SILT (SP-SM)** dense, wet, brown with gray mottles, fine to medium, some coarse, fine to coarse gravel	SP-SM	31	⊠				
-22.0	45	▨	**SILTY CLAY (CL)** stiff, wet, brown, trace fine to medium sand, low plasticity	CL	18	⧓	21	109		○ △
-25.5		▨	**SANDY CLAY (CL)** stiff, wet, brown, fine to medium, low plasticity	CL	23	⧓				
-28.0	50		Bottom of Boring at 50 feet							
	55									
	60									

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,985,247

Easting: 6,153,484

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-12

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-30-01 FINISH DATE: 4-30-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 51.5 FT.

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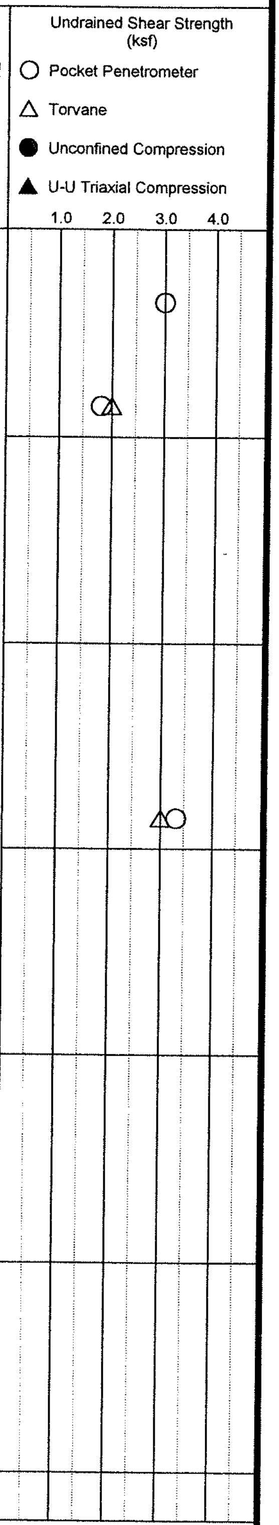
ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
22	0	SURFACE ELEVATION: 22 FT. (+/-) **SILTY CLAY (CL) [FILL]** very stiff, moist, brown to dark brown with black mottles, some fine to coarse sand, some fine to coarse gravel, low to moderate plasticity	CL	34	12	117	
19.0		**SILTY CLAY (CL) [FILL]** stiff, moist, dark brown with brown and orange mottles, some fine to medium sand, trace fine gravel, low to moderate plasticity (Moisture/Density-Porosity Test)	CL	21	30	99	
14.8		**SANDY CLAY (CL) [FILL]** stiff, dark brown with orange and light brown mottles, fine to coarse sand, low plasticity increasing coarse sand, increasing fine gravel	CL				
13.3		**SILTY CLAY (CH)** stiff to very stiff, moist, dark brown to black, high plasticity	CH	10 30	 26	 96	
5.5		**SILTY SAND (SM)** loose, wet, blue and orange mottled, fine to medium sand, some clay (Particle Size Test)	SM	9 8	24	101	 30
-6.0		**SAND (SP)** loose to medium dense, wet, gray to bluish-gray, fine to coarse	SP				
-8.0	30	*Continued Next Page*					



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,986,457
Easting: 6,153,001

LA_CORP.GDT 7/23/01 MV* FLL

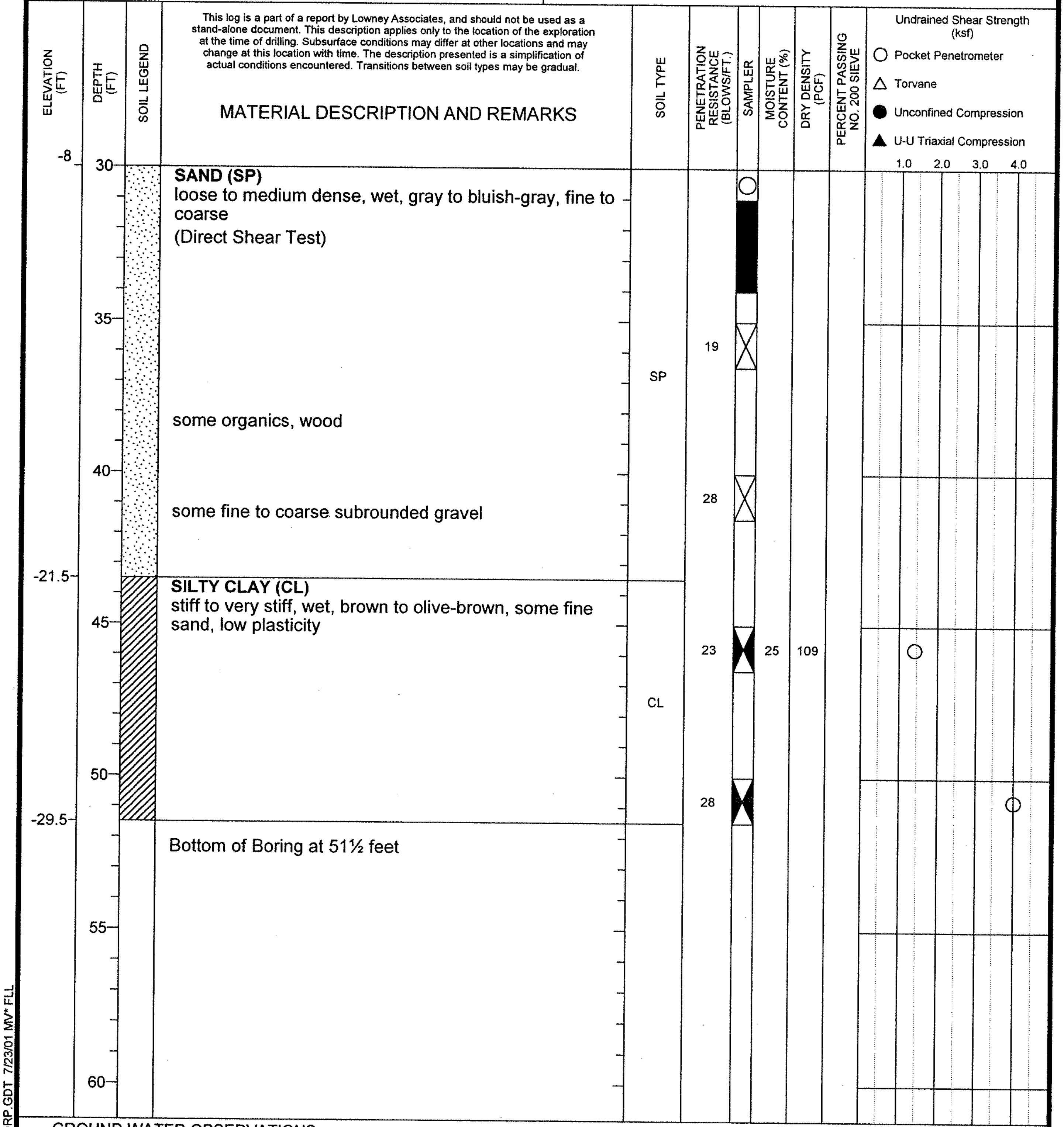
LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-12 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-30-01 FINISH DATE: 4-30-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 51.5 FT.



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SAND (SP)
loose to medium dense, wet, gray to bluish-gray, fine to coarse
(Direct Shear Test)

some organics, wood

some fine to coarse subrounded gravel

SILTY CLAY (CL)
stiff to very stiff, wet, brown to olive-brown, some fine sand, low plasticity

Bottom of Boring at 51½ feet

Soil type	Penetration resistance (blows/ft.)	Moisture content (%)	Dry density (pcf)
SP	19		
	28		
CL	23	25	109
	28		

LA CORP.GDT 7/23/01 MV* FLL

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,986,457
Easting: 6,153,001

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-13

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 5-1-01 FINISH DATE: 5-2-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 40.0 FT.

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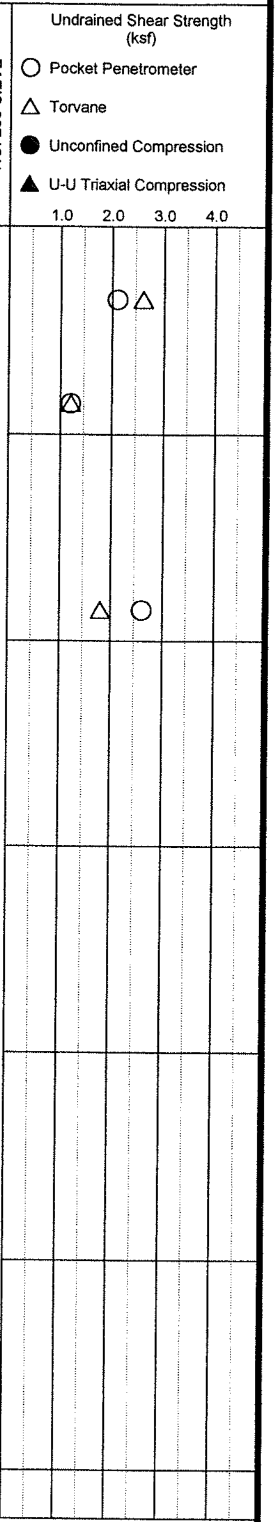
SURFACE ELEVATION: 12 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
12	0	**SILTY CLAY (CH)** stiff, moist, black, high plasticity	CH				
				14	29	92	
				11	39	81	
7.0	5	**SILTY CLAY (CL)** stiff, moist, dark gray to brown, trace fine sand, low plasticity	CL				
				19	24	104	
	10						
-1.0		**CLAYEY SAND (SC)** medium dense, wet, bluish gray and orange, fine to medium	SC				
-3.0	15	**SAND WITH SILT (SP-SM)** loose, wet, gray, fine to medium, trace coarse	SP-SM				
				8			
	20						
		medium dense		26	20		12
	25						
				28			
-18.0	30	*Continued Next Page*					



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,986,322
Easting: 6,152,924

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-13 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 5-1-01 FINISH DATE: 5-2-01	COMPLETION DEPTH: 40.0 FT.

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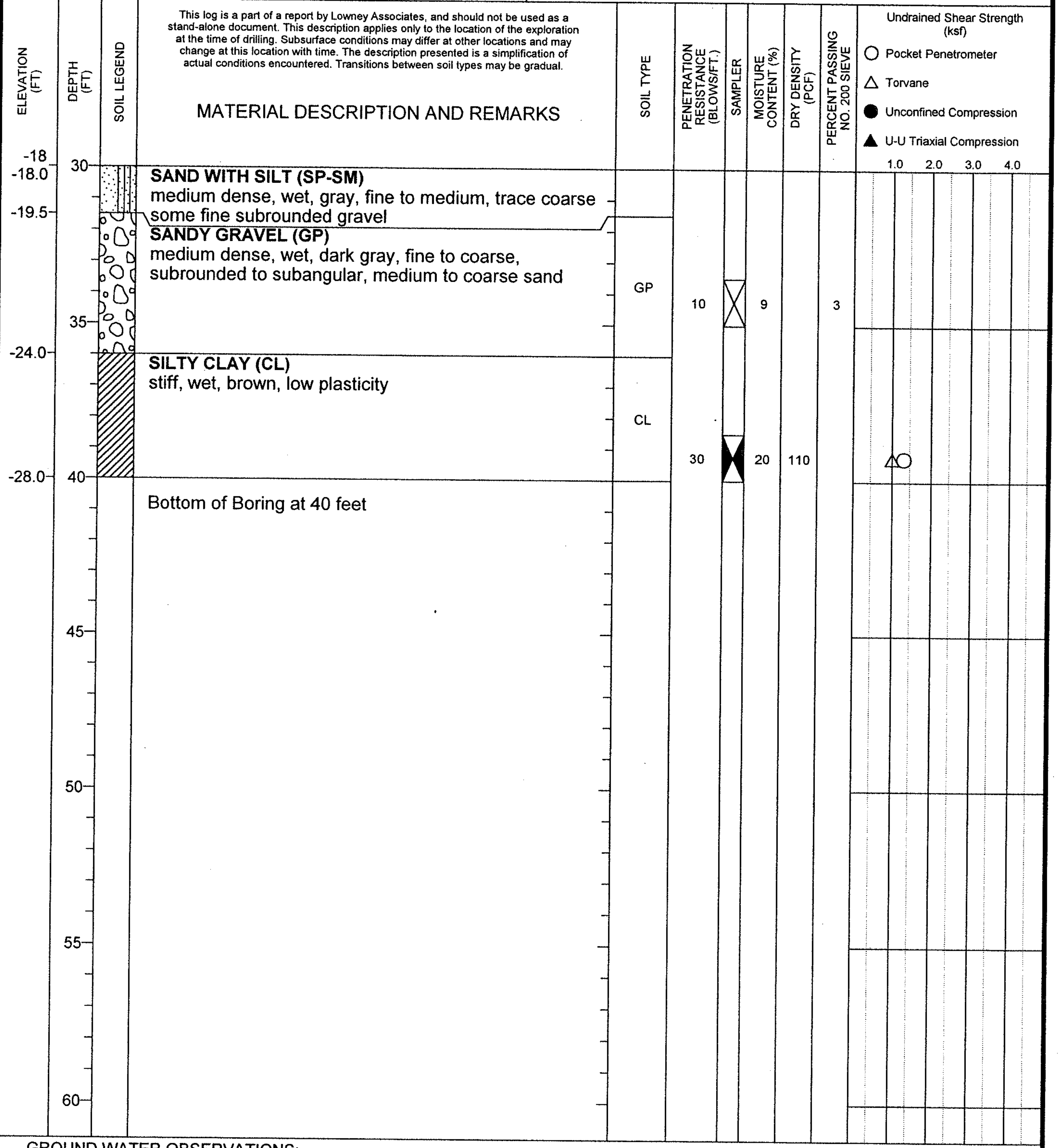

Undrained Shear Strength (ksf)

○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-18 -18.0	30	**SAND WITH SILT (SP-SM)** medium dense, wet, gray, fine to medium, trace coarse some fine subrounded gravel					
-19.5		**SANDY GRAVEL (GP)** medium dense, wet, dark gray, fine to coarse, subrounded to subangular, medium to coarse sand	GP	10	9		3
	35						
-24.0		**SILTY CLAY (CL)** stiff, wet, brown, low plasticity	CL	30	20	110	
-28.0	40	Bottom of Boring at 40 feet					
	45						
	50						
	55						
	60						

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,986,322
Easting:	6,152,924

LA CORP.GDT 7/23/01 MV* FLL

EXPLORATORY BORING: EB-14

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 5-2-01 FINISH DATE: 5-2-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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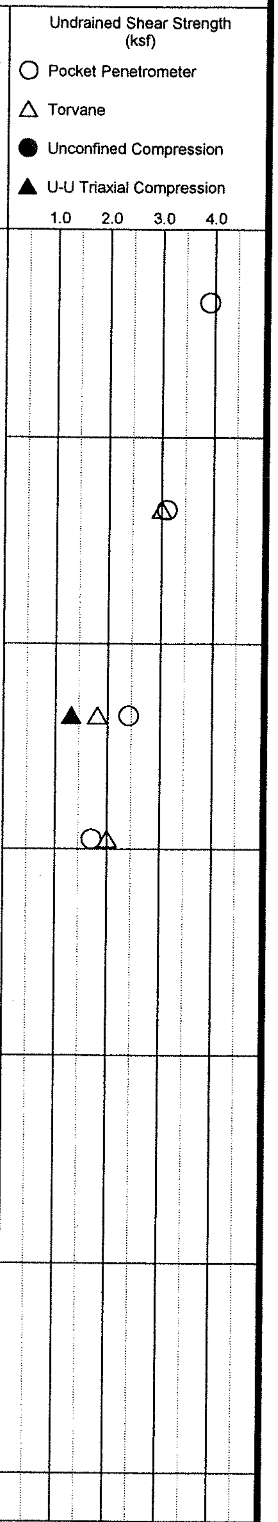
SURFACE ELEVATION: 19 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
19	0	**SILTY CLAY (CL) [FILL]** very stiff, damp, brown with black mottles, some fine to coarse sand, some fine gravel, moderate plasticity Plasticity Index = 27, Liquid Limit = 44	CL	31	17	113	
				29	25	101	
11.5		**SILTY CLAY (CH)** stiff, moist, black, high plasticity	CH	13	35	85	
				17	30	97	
4.0	15	**SILTY CLAY (CL)** stiff, moist, light brown to olive-brown, some fine sand, low plasticity (CU-Triaxial Test)	CL				
1.0		**SILTY SAND (SM)** loose to medium dense, wet, olive-brown with orange mottles, fine, trace organics	SM	10	25	100	
				11	25		24
-8.0		**SAND WITH SILT (SP-SM)** dense, wet, gray, medium to coarse, some fine, some fine gravel	SP-SM				
-11.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,986,727
Easting: 6,152,375

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

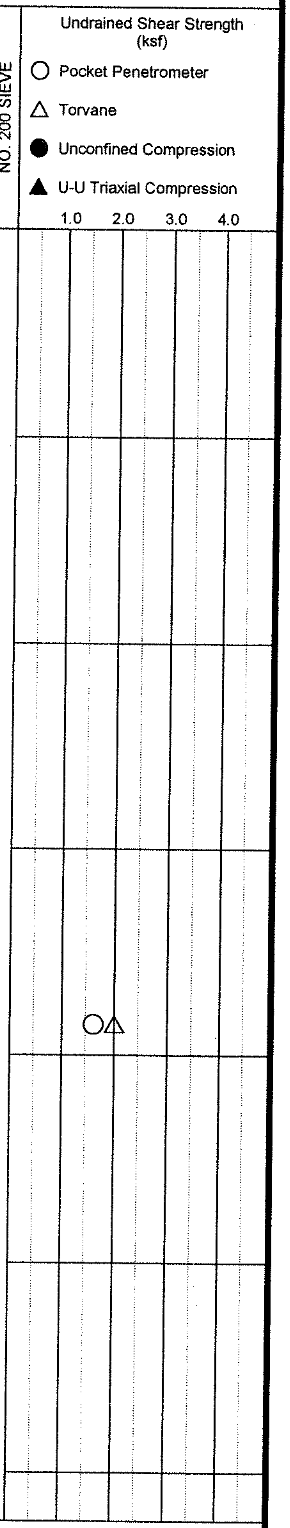
EXPLORATORY BORING: EB-14 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 5-2-01 FINISH DATE: 5-2-01	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-11	30	**SAND WITH SILT (SP-SM)** dense, wet, gray, medium to coarse, some fine, some fine gravel	SP-SM				
	35			38			
	40			42	8		7
-24.0		**SILTY SAND (SM)** medium dense, wet, olive-brown, fine	SM	13			
	45						
-28.0		**SILTY CLAY (CL)** stiff, wet, olive to brown, low plasticity	CL	30	19	109	
-31.0	50	Bottom of Boring at 50 feet					
	55						
	60						



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,986,727
Easting: 6,152,375

LA_CORP.GDT 7/23/01 MV*FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-14A

Sheet 1 of 1

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-26-01 FINISH DATE: 4-26-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 18.5 FT.

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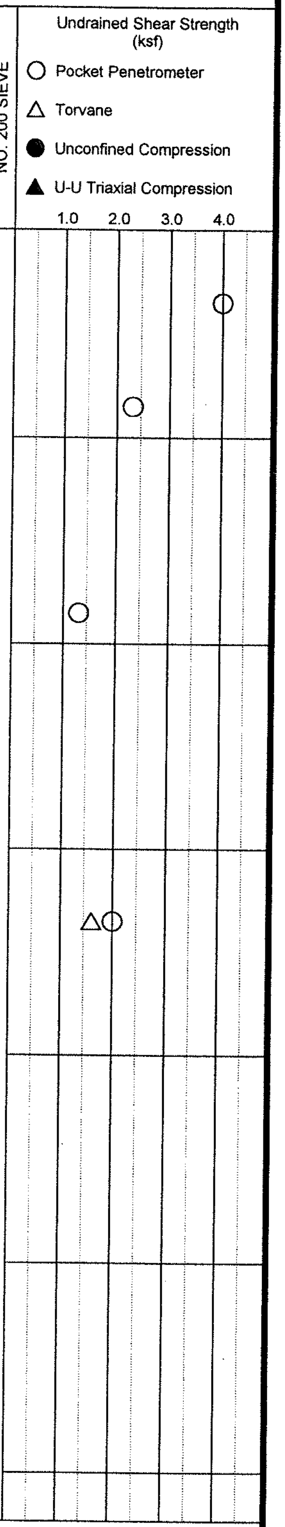
MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
SILTY CLAY (CL) [FILL] stiff, damp, dark brown with light brown mottles, some fine to coarse sand, some fine gravel, low plasticity	FILL, CL	36	16	115	
SILTY CLAY (CL) stiff, moist, olive to light brown, some fine to medium sand, low plasticity	CL	34	25	105	
SAND (SP) damp, fine to medium	SP				
SILTY CLAY (CH) stiff, moist, black, high plasticity	CH	4	32		
SANDY CLAY (CL) stiff, moist, orangish brown with gray mottles, some fine to medium sand, low plasticity	CL	21	26	100	
Bottom of Boring at 18½ feet discontinued drilling due to strange odor					



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-15

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-25-01 FINISH DATE: 4-25-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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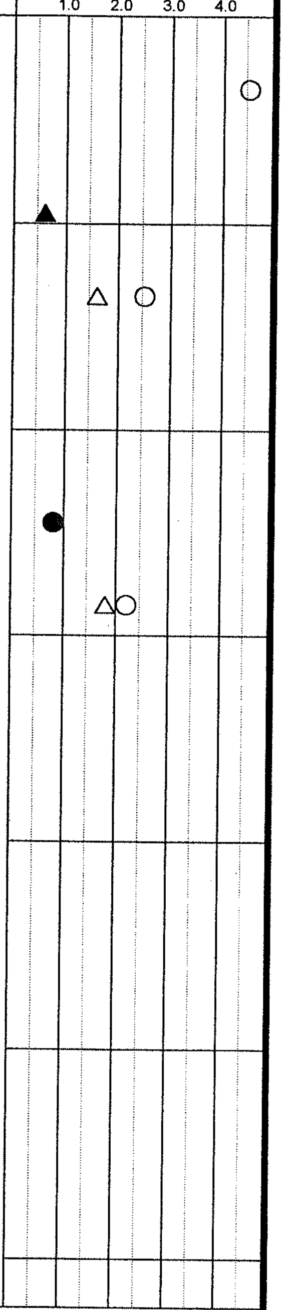
ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
19	0	SURFACE ELEVATION: 19 FT. (+/-) 6 inches aggregate base					
18.5		**SILTY CLAY (CL-CH) [FILL]** very stiff, damp to moist, dark brown and brown mottled, some fine sand, moderate to high plasticity Plasticity Index = 21, Liquid Limit = 39	CL-CH	31	16	110	
		(Particle Size Test, R-Values) Plasticity Index = 35, Liquid Limit = 56 (CU Triaxial Test, Hydraulic Conductivity Test)					
	5						86
12.0		**SILTY CLAY (CH)** stiff, wet, light-brown, trace fine sand, high plasticity	CH	19	29	90	
	10	(CU-PP Triaxial Test) Pasticity Index = 34, Liquid Limit = 51					
6.0		**SANDY CLAY (CL)** stiff, wet, light brown, fine sand, trace coarse sand, low plasticity	CL	13	20	110	
	15						
3.0		**CLAYEY SAND (SC)** medium dense, wet, olive, fine to medium	SC				
-1.0	20	**SILTY SAND (SM)** medium dense, wet, gray, fine to medium	SM	19	21		28
	25			13			
-9.5			SP				
-11.0	30	*Continued Next Page*					

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,987,457
Easting:	6,152,092

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

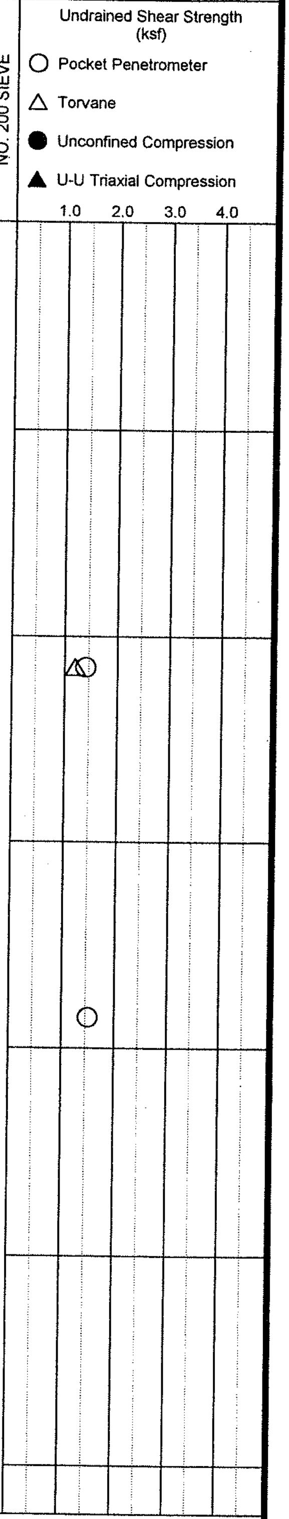
EXPLORATORY BORING: EB-15 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 50	PROJECT NO: 1697-1
BORING TYPE: 3-7/8 INCH ROTARY WASH	PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOGGED BY: BAH	LOCATION: MILPITAS,CA
START DATE: 4-25-01 FINISH DATE: 4-25-01	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-11	30	**GRAVELLY SAND (SP)** dense, wet, gray-brown, fie to coarse, fine to coarse gravel	SP	44			
	35			36			
-20.5	40	**SILTY CLAY (CL)** stiff, wet, dark olive, some fine sand, low plasticity	CL	20	21	106	
-26.0	45	**SANDY SILT** stiff, wet, brown to olive, fine sand, low plasticity	ML	20			
				19	24	102	56
-31.0	50	Bottom of Boring at 50 feet					
	55						
	60						



GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,987,457

Easting: 6,152,092

LA_CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-16

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-25-01 FINISH DATE: 4-25-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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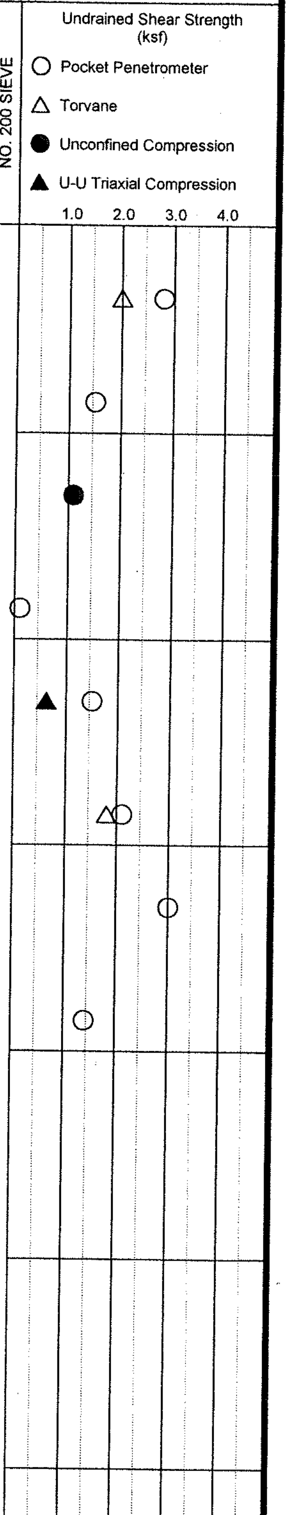
SURFACE ELEVATION: 18 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
18	0	6 inches aggregate base	AB				
17.5		**SILTY CLAY (CL) [FILL]** stiff, moist, brown, some fine sand, moderate plasticity Plasticity Index = 23, Liquid Limit = 38	CL	19	23	101	
15.0	5	**SILTY CLAY (CL) [FILL]** stiff, moist, dark brown, some fine sand, high plasticity Plasticity Index = 37, Liquid Limit = 55 (CU-PP Triaxial Test)	CH	19	26		
9.8	10	**SANDY SILT (ML)** medium stiff, wet, olive-brown fine sand, low plasticity	ML	12	30	87	
7.0	15	**SANDY CLAY (CL)** stiff, wet, light brown to brown, fine to coarse sand, trace fine gravel, low plasticity (CU Triaxial Test)	CL	19	17	114	
-0.5	20	**CLAYEY SILT (ML)** stiff, wet, reddish brown with blue-gray mix, low plasticity	ML	19	26	101	
-4.5	25	**SAND WITH SILT (SP-SM)** medium dense, wet, dark olive, fine to medium	SP-SM	19 13	25	97	
-12.0	30						



Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,987,852
Easting: 6,151,795

LA_CORP.GDT 7/23/01 MV*FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-16 Cont'd

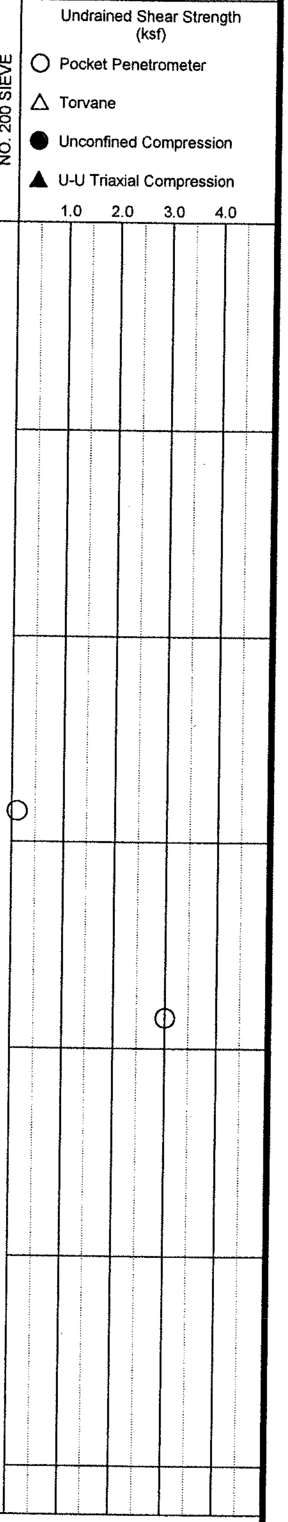
Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-25-01 FINISH DATE: 4-25-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-12	30	**SAND WITH SILT (SP-SM)** medium dense, wet, dark olive, fine to medium	SP-SM					
-14.5		**GRAVELLY SAND (SP)** dense, wet, gray-brown, medium to coarse, some fine, fine to coarse gravel	SP	33	☒			
	35							
				14	☒			
-22.0	40	**SILTY CLAY (CL)** medium stiff to stiff, wet, light brown, some fine sand, some fine gravel, low plasticity	CL					
	45			15	⧗	22	102	
				24	⧗			
-32.0	50	Bottom of Boring at 50 feet						
	55							
	60							



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,987,852
Easting:	6,151,795

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-17

Sheet 1 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-24-01 FINISH DATE: 4-24-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

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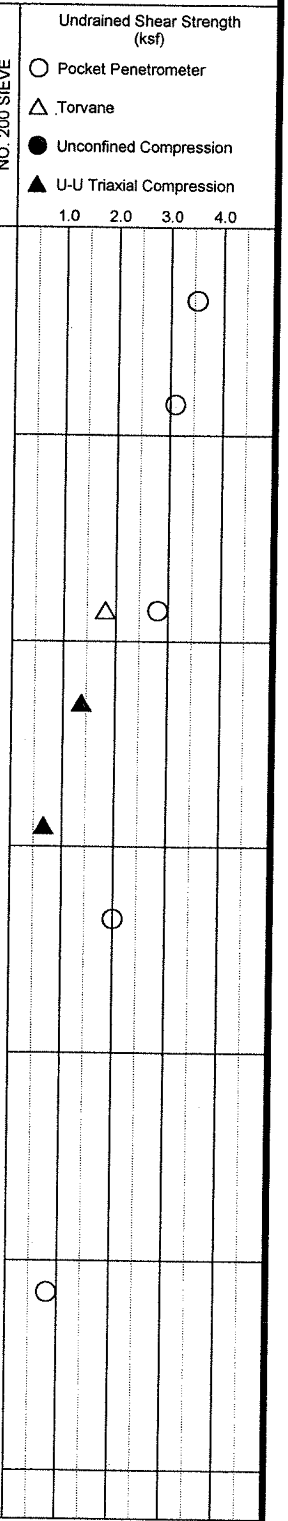
MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 20 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
20	0						
19.5		6 inches aggregate base	AB				
		SILTY CLAY (CL) [FILL] stiff, damp, brown with orange mottles, some sand, some fine gravel, low plasticity	CL	31	13	113	
15.5				23	19	113	
15.0	5	**SILTY CLAY (CH)** stiff, moist, dark gray-brown to black, some fine sand, trace fine gravel high plasticity	CH				
		SANDY CLAY (CL) very stiff, moist, dark gray, fine sand, moderate plasticity Plasticity Index = 19, Liquid Limit = 39 (Consolidation Test, Hydraulic Conductivity Test)	CL	23	22	101	
	10						
8.5		**SILTY CLAY (CL)** stiff, wet, dark gray, some fine sand, low plasticity (Hydraulic Conductivity Test, CU Triaxial Tests)	CL				
	15						
4.0		**SANDY CLAY (CL)** stiff, wet, olive-brown to light brown, fine sand, low plasticity	CL	17			
0.0	20	**CLAYEY SAND (SC)** wet, loose, light brown, fine sand	SC	10	22	105	
-3.0		**CLAYEY SILT (ML)** medium stiff, wet, light brown to olive-brown, fine sand, low plasticity	ML	11			
	25						
-10.0	30						



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GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing:	1,987,967
Easting:	6,151,187

LA CORP.GDT 7/23/01 MV* FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-17 Cont'd

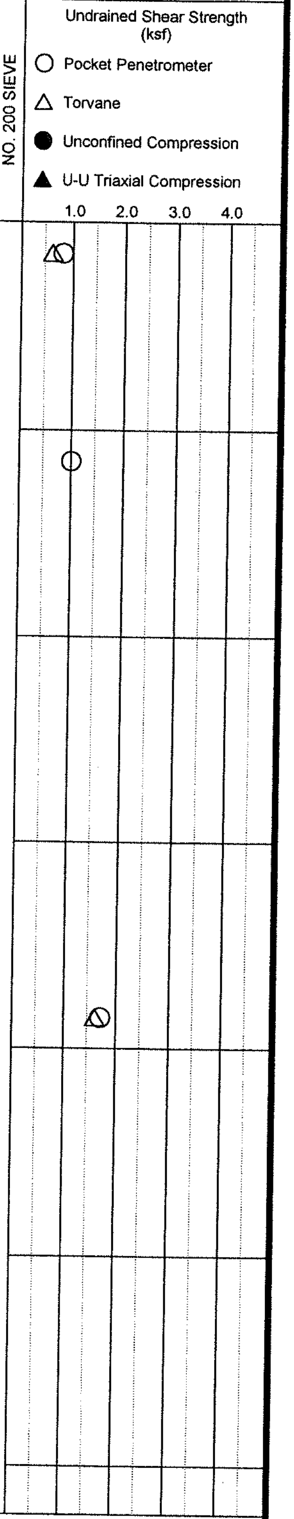
Sheet 2 of 2

DRILL RIG: FAILING 50
BORING TYPE: 3-7/8 INCH ROTARY WASH
LOGGED BY: BAH
START DATE: 4-24-01 FINISH DATE: 4-24-01

PROJECT NO: 1697-1
PROJECT: BERRYESSA CREEK LEVEES PROJECT
LOCATION: MILPITAS,CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-10	30	▥	**CLAYEY SILT (ML)** medium stiff, wet, light brown to olive-brown, fine sand, low plasticity	ML	11	⧗	28	92	
	35				13	⧗			
-18.5		░	**GRAVELLY SAND (SP)** medium dense, wet, gray-brown, medium to coarse, some fine, fine to coarse gravel	SP	36	☒			
	40				18	☒			
-24.5	45	▨	**SILTY CLAY (CL)** stiff, wet, light brown, trace fine sand, low plasticity	CL	27	⧗	29	96	
-30.0	50		Bottom of Boring at 50 feet						
	55								
	60								



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,987,967
Easting: 6,151,187

LA_CORP.GDT 7/23/01 MV*FLL

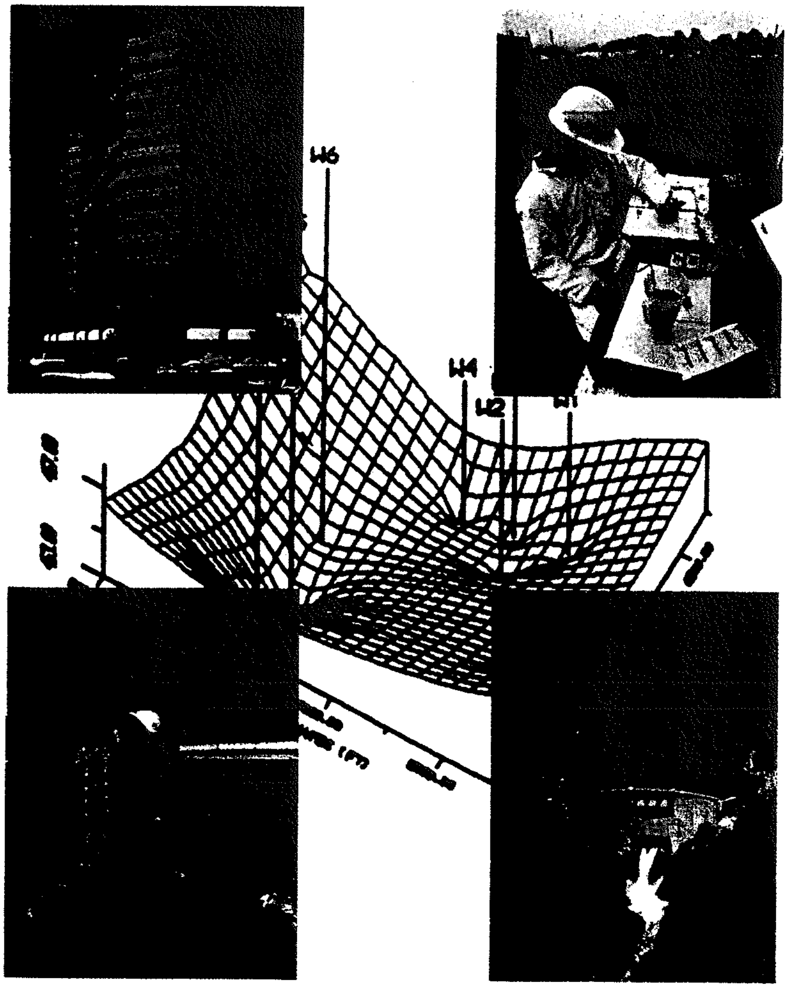
LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

HARZA

Consulting Engineers and Scientists

Geotechnical Engineering Analysis Report
Berryessa Creek Levee
Milpitas, California

Project No. M053-G
May _, 1998



Geotechnical Engineering Analysis Report
Berryessa Creek Levee
Milpitas, California

Project No. M053-G
May _, 1998

Prepared For:

Nolte and Associates, Inc.
One North First Street, Suite 450
San Jose, California 95113

Prepared By:

Harza Engineering Company
425 Roland Way
Oakland, CA 94621

Albert S. Wang, Ph.D., P.E.
Senior Project Engineer

Ronald L. Bajuniemi, P.E., G.E.
Chief Geotechnical Engineer

M053GRPT.001
05/18/98

APPENDIX A
Exploratory Boring Logs

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	○○	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	●	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	▤	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	▨	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	●	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	⁘	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	▥	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	▨	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMITS 50% OR LESS		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	▨	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	┆	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMITS GREATER THAN 50%		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	▨	ORGANIC SILTS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			Pt	≈	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION - ASTM D2487 - 85

Perm — Permeability
Consol — Consolidation
LL — Liquid Limit (%))
PI — Plastic Index (%)
G_s — Specific Gravity
MA — Particle Size Analysis
■ — "Undisturbed" Sample
☒ — Bulk or Classification Sample

	Shear Strength (psf)	Confining Pressure	
TxUU (FM) or (S)	3200	(2600)	Unconsolidated Undrained Triaxial Shear (field moisture or saturated)
TxCU (P)	3200	(2600)	Consolidated Undrained Triaxial Shear (with or without pore pressure measurement)
TxCD	3200	(2600)	Consolidated Drained Triaxial Shear
SSCU (P)	3200	(2600)	Simple Shear Consolidated Undrained (with or without pore pressure measurement)
SSCD	3200	(2600)	Simple Shear Consolidated Drained
DSCD	2700	(2000)	Consolidated Drained Direct Shear
UC	470		Unconfined Compression
LVS	700		Laboratory Vane Shear

KEY TO TEST DATA

Soil Classification and Test Data Key

PLATE

nta Clara Valley Water District

Drawn	Job Number	Approved	Date	Revised	Date
AGREDA					

E: G:\DWGS\SDSKPROJ\4018_010\SHEETS\DATA_KEY\DATA_KEY.DWG

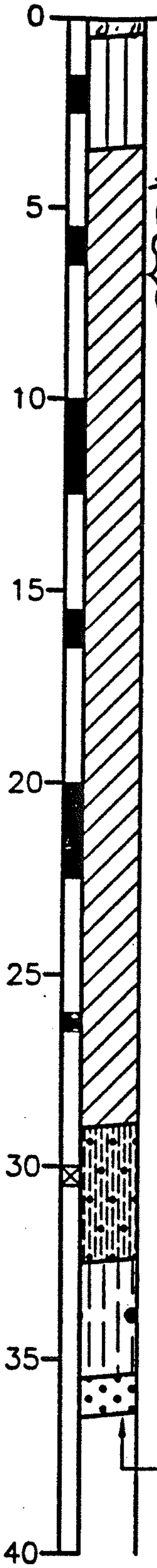
DRAFT

DRAFT

Equipment Rotary Wash

Top Elevation 15.65 Feet Date 10/02/97

Laboratory Tests	Blows/foot 14	Moisture Content %	Dry Density (psf)	Depth (ft)	Description
				0	6" Aggregate Base
	8				Brown Sandy Silt (ML) Medium Stiff (Fill)
	12	--		5	DarK ~~Light~~ Brown Silty Clay (CL) Stiff (Fill)
	Push			10	Brown Silty Clay (CL) Stiff (Fill)
	4			15	
	Push			20	Blue Gray Fat Clay (CL) Moist Soft
	10			25	
	Push			30	Gray Silty Sand (SM) Medium Dense
					Gray and Brown (GM) (GP) Fine Sandy Gravel
	34			35	Gray Sand (SP) Medium To Coarse, Medium Dense
					Bottom of Boring at 36.5 Feet
				40	



DRAFT

LOG OF BORING B-1

BERRYESSA CREEK LEVEE

Milpitas, California ← Bold

PLATE

Santa Clara Valley Water District

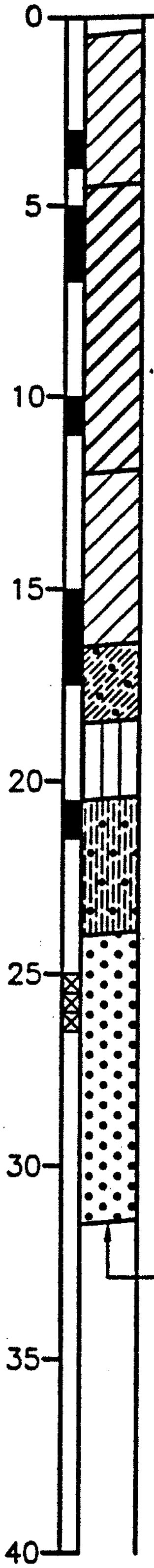
Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

F: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERR-01A.DWG

Equipment Rotary Wash

Top Elevation 16.71 Feet **Date** 10/02/97

Laboratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)
	10		
	Push	--	
	10		
	Push		
	10		
	18		
	24		



6" Aggregate Base

Gray Brown Silty Clay (CL)
With Trace Of Gravel, Stiff (Fill)

Light Brown Fat Clay (CH)
Moist, Stiff (Fill)

Gray Fat Clay (CH)
Moist, Stiff (Fill)

Light Brown Silky Clay (CL)
Moist, Stiff

Gray Brown Clayey Sand (SC)

Light Brown Sandy Silt (ML)

Fined Silty Sand (SM)
Fine Grained Medium Coarse

Gray Brown Sand (SP)
Medium Dense
Grading to Coarser Sand
Dense Of Gravel

Bottom of Boring at 31.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-2

BERRYESSA CREEK LEVEE

Milpitas, California

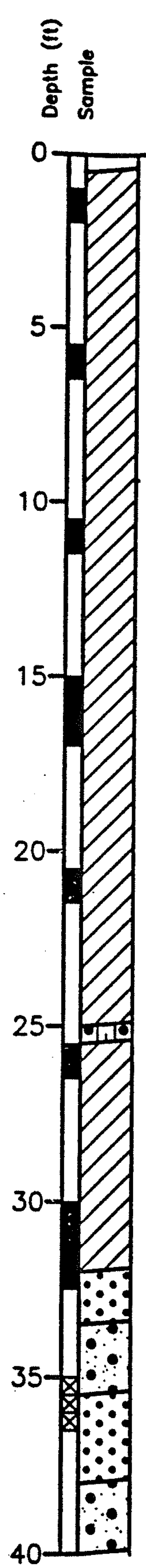
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Equipment Rotary Wash

Top Elevation 17.6 Feet **Date** 10/03/97

Laboratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)	Description
	24			6" Aggregate Base Gray Brown Silty Clay (CL) With Trace Gravel, Very Stiff (Fill)
	12	--		Light Brown Silty Clay (CL) With Some Fine Sand Stiff (Fill)
	18			Dark Brown Silty Clay (CL) Very Stiff (Fill)
	Push			Light Brown Silty Fat Clay (CH) Stiff
	20			Gray Brown Silty Clay (CL) Very Stiff
				Gray Brown Silty Clay (CL) Soft
	5			Gray Fine Silty Sand (SM)
				Gray Brown Silty Clay (CL) Soft To Medium, Stiff
	Push			Gray Sand (SP) Medium Dense
	35			Gray Fine Sandy Gravel (GP) Subangular To Angular Grain
				Gray Sand (SP) Coarse Grained Dense
				Gray Gravel (GP)



Santa Clara Valley Water District

LOG OF BORING B-3

BERRYESSA CREEK LEVEE

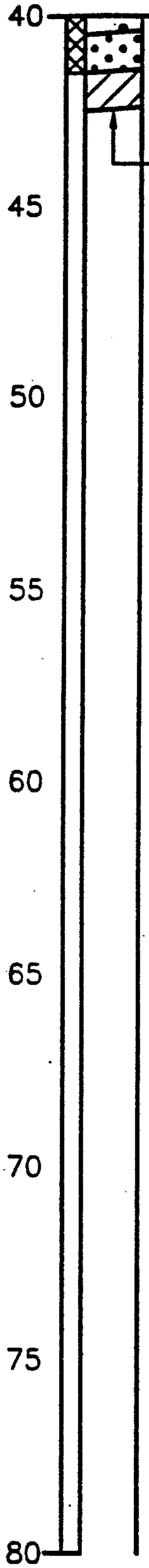
Milpitas, California

PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/14/98		

LE: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-03A.DWG

aboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)	Sample	(Continuation of Log)
	18				Gray Sand (SP), Medium Dense Brown Silty Clay (CL) Stiff Bottom of Boring at 42.5 Feet
		--			



ʋanta Clara Valley Water District

LOG OF BORING B-3

BERRYESSA CREEK LEVEE

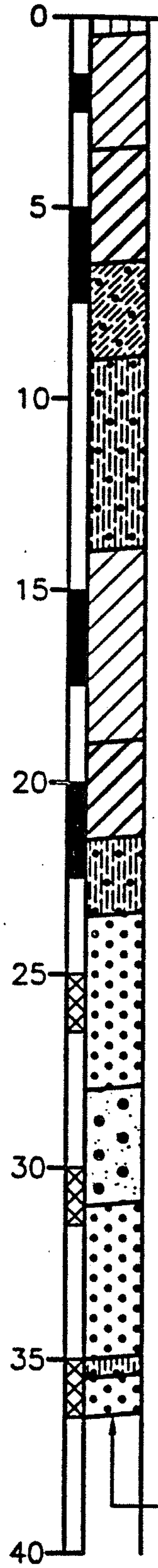
Milpitas, California

PLATE

Equipment Rotary Wash

Top Elevation 10.8 Feet Date 10/03/97

Laboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	Push		
	Push	--	
	Push		
	Push		
	28		
	25		
	37		



Brown Silt (ML), Soft

Dark Bown Silty Clay (CL)
Stiff

Dark Gray Fat Clay (CH)

Brown Clayey Sand (SC)

Silty Gray Sand (SM)
With Some Fine Gravel Medium Dense

Brown Silty Clay (CL)

Brown And Blue Fat Clay (CH)

Gray Silty Sand (SM)
Fine Grained, Medium Dense

Gray Fine To Coarse Sand (SP)
With Some Gravel, Dense

Gray Fine Gravel (GP)
Medium Dense

Gray Sand (SP)
Medium To Coarse Grained, Dense

Gray Silty Sand (SM)
Fine Grained, Dense

Gray Coarse Sand (SP)
Dense

Bottom @ 36.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-4

BERRYESSA CREEK LEVEE

Milpitas, California

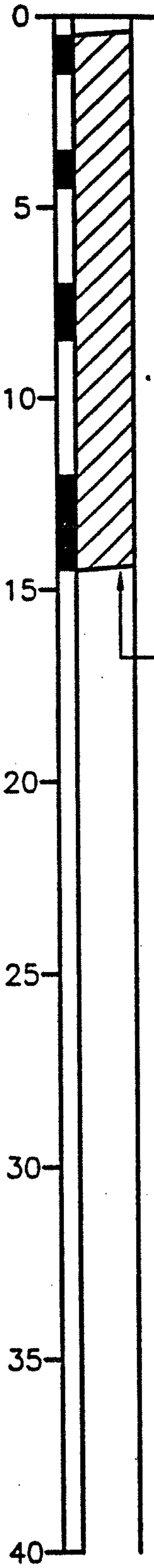
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Equipment Rotary Wash

Top Elevation 16.3 Feet Date 10/06/97

aboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	24		
	15		
		--	
	21		
	Push		



6" Aggregate Base

Brown Silty Clay (CL)
Very Stiff (Fill)

Dark Brown Silty Clay (CL)
Stiff (Fill)

Brown Silty Clay (CL)
Soft

Bottom @ 14.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-5

BERRYESSA CREEK LEVEE

Milpitas, California

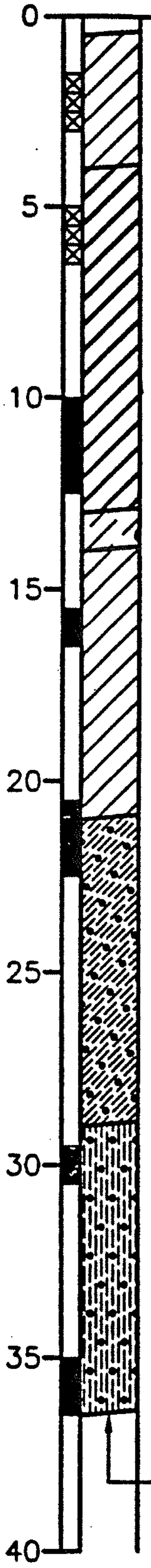
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Equipment Rotary Wash

Top Elevation 16.2 Feet **Date** 10/06/97

oratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	19		
	14	--	
	Push		
	14		
	Push		
	38		
	Push		
	38		
	21		



6" Aggregate Base

Dark Brown Fat Clay (CL
With Some Gravel, Very Stiff (Fill)

Dark Brown Fat Clay (CH)
With Some Fine Sand, Stiff (Fill)

Clayey Gravel (GC), With Wood Debries

Light Brown Clay (CL)

Brown Silty Clay (CL)
Stiff

Dark Brown Clayey Sand (SC)

Gray Silty Sand (SM)
Dense

Bottom @ 36.5 Feet

anta Clara Valley Water District

LOG OF BORING B-6

BERRYESSA CREEK LEVEE

Milpitas, California

PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

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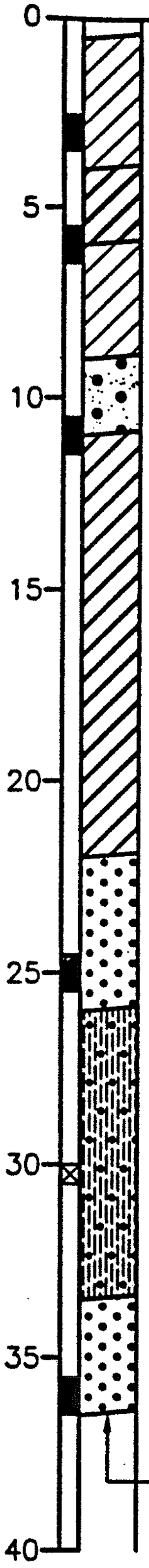
Equipment Rotary Wash

Top Elevation 18.8 Feet **Date** 10/07/97

aboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	15		
	12	--	
	7		
	14		
	20		
	24		



6" Aggregate Base

Brown Silty Clay (CL)
Stiff (Fill)

Gray Silty Fat Clay (CH)
(Fill)

Brown Sandy Clay (CL)
With Same Gravel, Stiff (Fill)

Brown and Gray Gravel (GP)
Loose (Fill)

Dark Brown Silty Clay (CH)
Soft

Brown Sand (SP)
Fine Grained Medium Dense

Light Brown Silty Sand (SM)
Medium Dense

Gray Sand (SP)
Medium Dense
Medium to Coarse Grained
With Some Gravel

Bottom @ 36.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-7

BERRYESSA CREEK LEVEE

Milpitas, California

PLATE

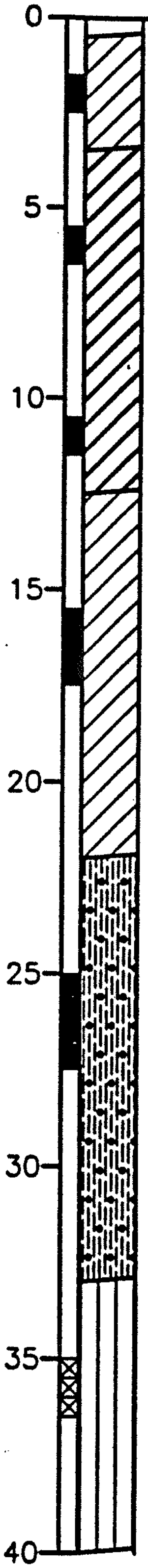
Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

FILE: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-07A.DWG

Equipment Rotary Wash

Top Elevation 20.9 Feet **Date** 10/07/97

Laboratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)
	17		
	18	--	
	17		
	7		
	Push		
	18		



6" Aggregate Base
Brown and Gray Silty Clay (CL)
Pieces of Wood, Chips and Roots
With Gravel, Stiff (Fill)

Dark Brown Fat Silty Clay (CH)
Stiff With Roots (Fill)

Brown Silty Clay (CL)
Stiff, Getting Softer With Depth

Gray Silty Sand (SM)
Loose To Medium Dense

Light Brown Sandy Silt (ML)
Medium Stiff

Santa Clara Valley Water District

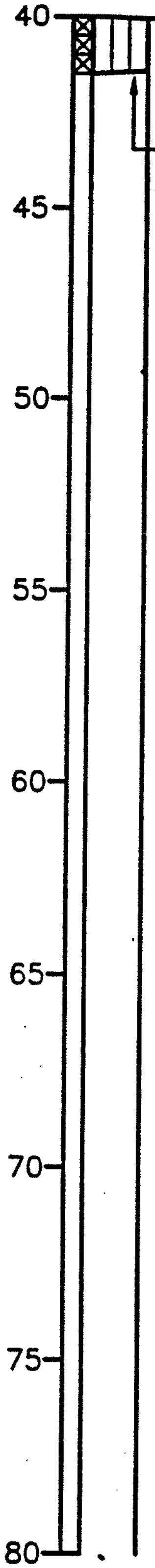
LOG OF BORING B-8
BERRYESSA CREEK LEVEE
Milpitas, California

PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Laboratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)
	7		
		--	

(Continuation of Log)



Light Brown Sandy Silt (ML)
Medium Stiff

Bottom @ 41.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-8
BERRYESSA CREEK LEVEE
Milpitas, California

PLATE

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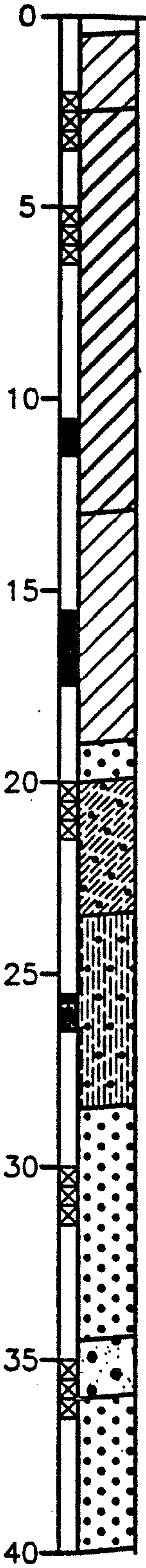
Equipment Rotary Wash

Top Elevation 18.4 Feet **Date** 10/08/97

Laboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	20		
	15	--	
	18		
	13		
	Push		
	14		
	25		
	39		



6" Aggregate Base
Light Brown Silty Clay (CL)
Stiff (Fill)

Dark Brown Fat Clay (CH)
Very Stiff (Fill)

Brown Sandy Clay (CL)
Stiff

Brown Sand (SP)
Coarse Grained, Medium Dense

Brown Clayey Sand (SC)
Fine Grained, Medium Dense

Brown Silty Sand (SM)
Fine Grained, Medium Dense

Gray Sand (SP)
Fine To Medium Grained, Medium Dense

Fine Gravel (GP)
With Little Sand Dense

Gray Sand (SP)
With Some Gravel,
Coarse Grained Dense

Santa Clara Valley Water District

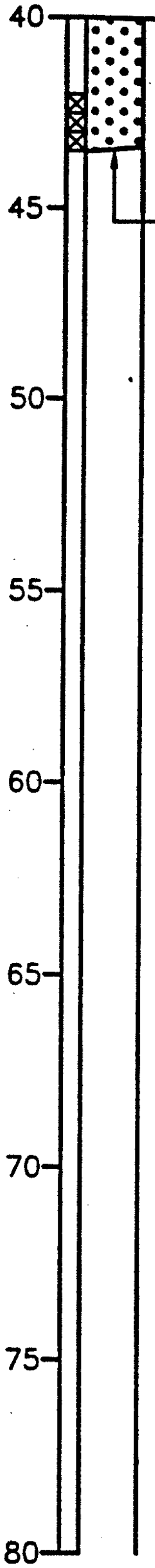
LOG OF BORING B-9
BERRYESSA CREEK LEVEE
Milpitas, California

PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

FILE: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-09A.DWG

Laboratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)	Depth (ft)	Sample	(Continuation of Log)
	33	--		40–43.5	☒	Gray Sand (SP) With Some Gravel, Coarse Grained Dense



Bottom @ 43.5 Feet

LOG OF BORING B-9

BERRYESSA CREEK LEVEE

Milpitas, California

PLATE

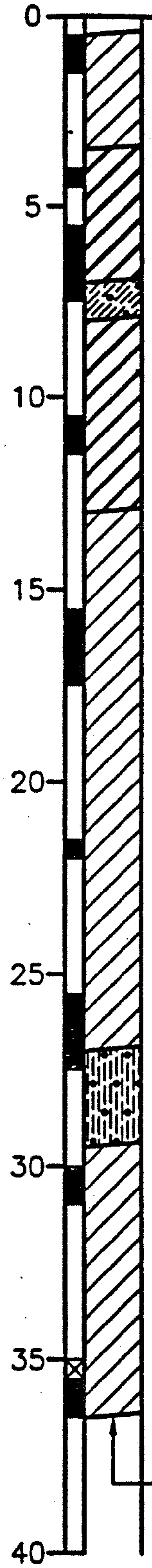
Santa Clara Valley Water District

FILE: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-09B.DWG

Equipment Rotary Wash

Top Elevation 20.77 Feet **Date** 10/08/97

Laboratory Tests	Blows/foot	Moisture Content ;	Dry Density (psf)	Depth (ft)	Description
	15			0	8" Aggregate Base
					Brown Silty Clay (CL) With Gravel Stiff (Fill)
	9				
	Push	--		5	Dark Brown Silty Fat Clay (CH) Medium Stiff (Fill)
					Brown Clayey Sand (SC) Fill
	17			10	Dark Brown Fat Clay (CH) Very Stiff (Fill)
	Push			15	
					Brown Silty Clay (CL) Medium Stiff To Stiff
				20	
	9				
	Push			25	
					Brown Silty Sand (SM) Fine Grained
	13			30	
					Brown Silty Clay (CL) Stiff To Very Stiff
	27			35	
					Bottom @ 36.5 Feet
				40	



Santa Clara Valley Water District

LOG OF BORING B-10

BERRYESSA CREEK LEVEE

Milpitas, California

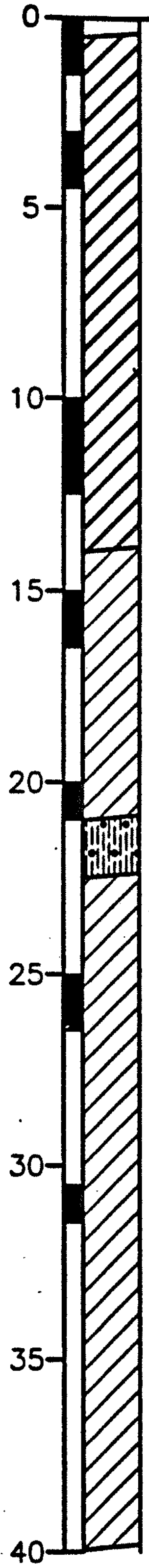
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Equipment Rotary Wash

Top Elevation 21.2 Feet **Date** 10/09/97

Laboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	23		
	19		
		--	
	Push		
	16		
	14		
	Push		
	17		



8" Aggregate Base

Dark Brown Fat Clay (CH)
Very Stiff (Fill)

Gray and Brown Fat Clay (CH)
Stiff

Brown Silty Clay (CL)
Very Stiff

Light Brown Silty Sand (SM)
Medium Dense

Brown Silty Clay (CL)
Stiff To Very Stiff

Brown Sandy Clay (CL)
Stiff

Santa Clara Valley Water District

LOG OF BORING B-11
BERRYESSA CREEK LEVEE
Milpitas, California

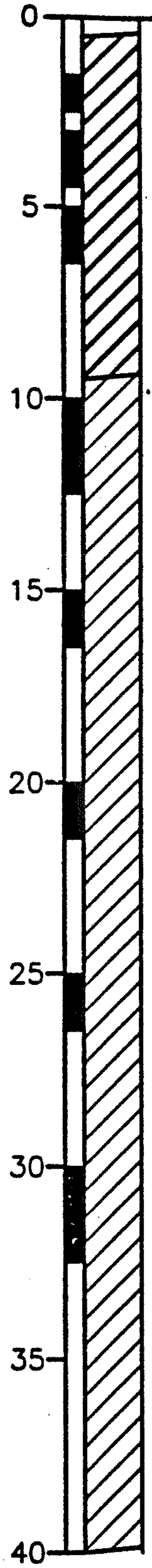
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

Equipme Rotary Wash

Top Elevation 21.7 Feet Date 10/09/97

oratory Tests	Blows/foot	Moisture Content %	Dry Density (psf)
	16		
	7		
	9	--	
	Push		
	14		
	21		
	19		
	37		



8" Aggregate Base

Dark Brown Fat Clay (CH)
Medium Stiff To Stiff (Fill)

Light Brown Clay (CL)
Stiff To Very Stiff

Tan Brown Silty Clay (CL)
Hard

anta Clara Valley Water District

LOG OF BORING B-12

BERRYESSA CREEK LEVEE

Milpitas, California

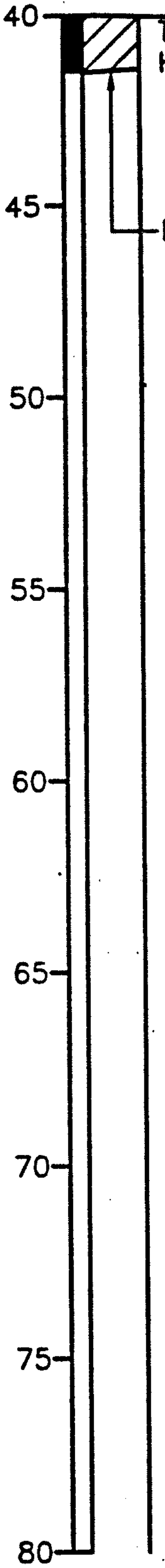
PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

FILE: G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-12A.DWG

(Continuation of Log)

aboratory Tests	Blows/foot	Moisture Content 7	Dry Density (psf)	Depth (ft)	Sample
32					



Tan Brown Silty Clay (CL)
Hard

Bottom @ 415 Feet

Santa Clara Valley Water District

LOG OF BORING B-12
BERRYESSA CREEK LEVEE
Milpitas, California

PLATE

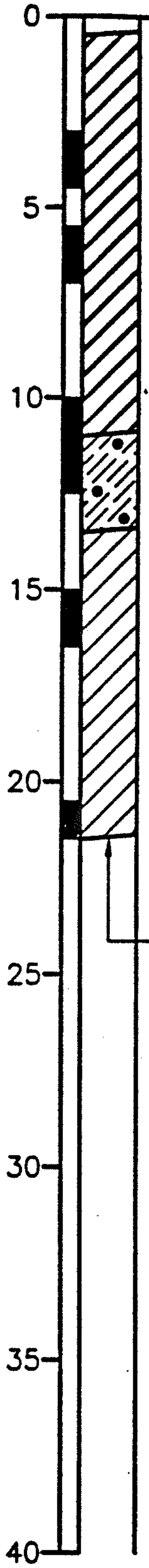
Equipment Rotary Wash

23.6 on site plan

Top Elevation 20.06 Feet **Date** 10/10/97

oratory Tests

Blows/foot	Moisture Content	Dry Density (psf)
11		
16	– –	
Push		
11		
22		



8" Aggregate Base

Dark Brown Fat Clay (CH)
Stiff To Very Stiff (Fill)

Light Brown Clayey Sand (SC)

Brown Sandy Clay (CL)
Stiff To Stiff

Bottom @ 21.5 Feet

anta Clara Valley Water District

LOG OF BORING B-13

BERRYESSA CREEK LEVEE

Milpitas, California

PLATE

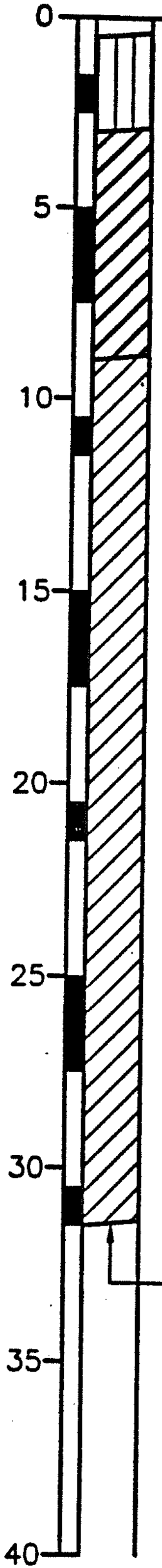
Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

G:\DWGS\SDSKPROJ\4018_010\SHEETS\BERB-13A.DWG

Equipment Rotary Wash

Top Elevation 22.54 Feet **Date** 10/16/97

Laboratory Tests	Blows/foot	Moisture Content	Dry Density (psf)
	31		
	Push	– –	
	14		
	Push		
	14		
	Push		
	20		



8" Aggregate Base
Brown and White Sandy Silt (ML)
Very Stiff (Fill)

Dark Brown Silty Fat Clay (CH)
Stiff (Fill)

Brown and White Silty Clay (CL)
Stiff

Light Brown (CL)
Sandy Clay and Moist
Stiff To Very Stiff

Bottom @ 31.5 Feet

Santa Clara Valley Water District

LOG OF BORING B-14
BERRYESSA CREEK LEVEE
Milpitas, California

PLATE

Drawn	Job Number	Approved	Date	Revised	Date
R. AGREDA	401801		01/13/98		

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APPENDIX C

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 75 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: Dry density determinations (ASTM D2937) were performed on 75 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Plasticity Index determinations (ASTM D4318) were performed on 15 samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 13 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Gradations: Eleven gradation tests (ASTM D422) were performed on samples of the subsurface soils. These tests were performed to assist in the classification of the soil and to determine grain size distributions of the soils. Results of these tests are presented in Appendix C.

Unconsolidated Undrained Shear Strength(UU): Sixteen triaxial unconsolidated undrained tests were performed to estimate the undrained shear strength for the "end of construction" condition or other short term load conditions. Results of these tests are presented in Appendix C.

Consolidated Undrained Shear Strength(CU): Seven triaxial consolidated undrained shear tests with pore pressure were performed measurements to estimate both undrained and drained shear strengths. Results of these tests are presented in Appendix C.

Hydraulic Conductivity: Ten samples were tested for hydraulic conductivity to aid in seepage analysis of the levee fill and foundation soils. Results of these tests are presented in Appendix C.

Consolidation: Consolidation tests (ASTM D2435) were performed on 6 undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility properties of these soils. Results of the consolidation tests are presented graphically in Appendix C.

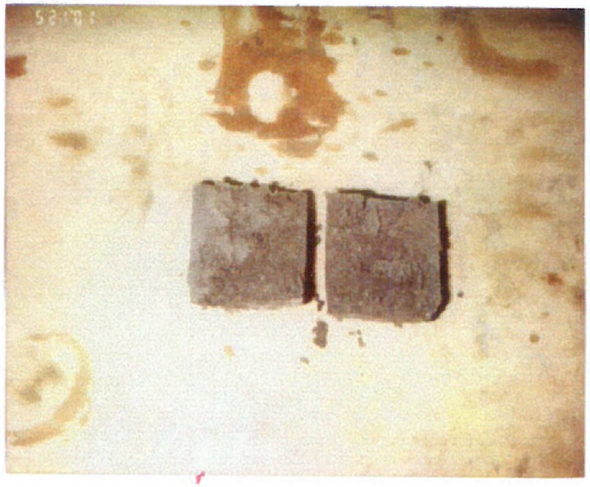
Compaction: Laboratory compaction tests (ASTM D1557) were performed on 5 representative samples of the surface soil to determine the maximum dry density and optimum moisture content. Results of the tests are presented in Appendix C.

Sieve and Hydrometer Analyses: Gradation and washed sieve analyses (ASTM D422 and D2217) were performed on four samples of the subsurface soils to aid in soil classification. Results of these tests are included in this appendix.

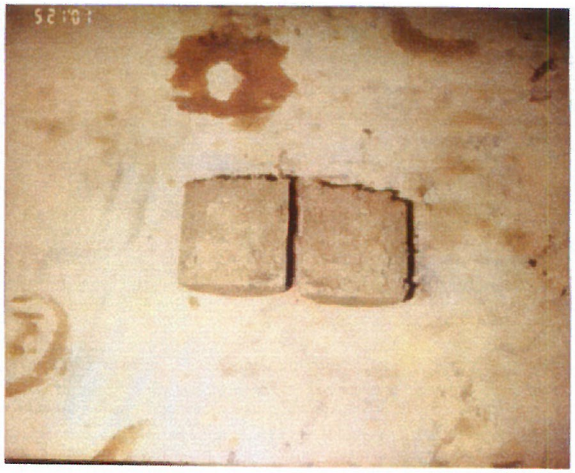
R-Values: R-value (resistance) tests (California Test Method No. 301) were performed on 3 representative samples of the surface soils at the site to provide data for pavement design. Detailed results of the tests are presented in the text of this report.

Geogrid Interaction Tests: Three bulk samples of the levee fill soils, representing the range of plasticity exhibited by the levee fill, are being tested with samples of Tensar geogrid materials to evaluate interaction coefficients. This testing was still in progress at the time of preparation of this report and will be provided as a supplement to this report when it is completed.

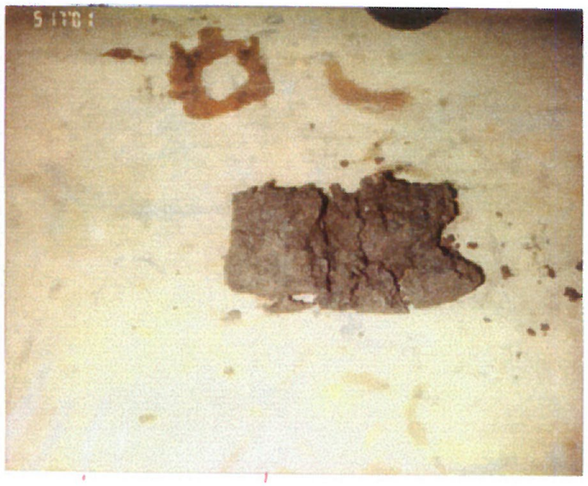
* * * * * * * * * * * * *



EB-1, 4A at 7-10 ft.



EB-2, 2A at 3-6 ft.



EB-3, 4A at 9-12 ft.

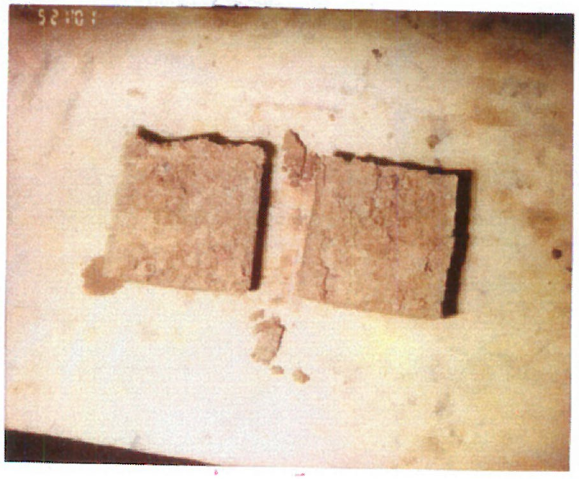
7/01*EB

HYDRAULIC CONDUCTIVITY TEST SAMPLES

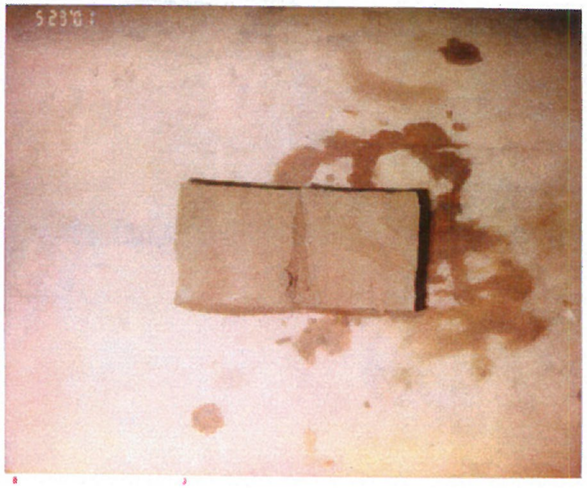
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

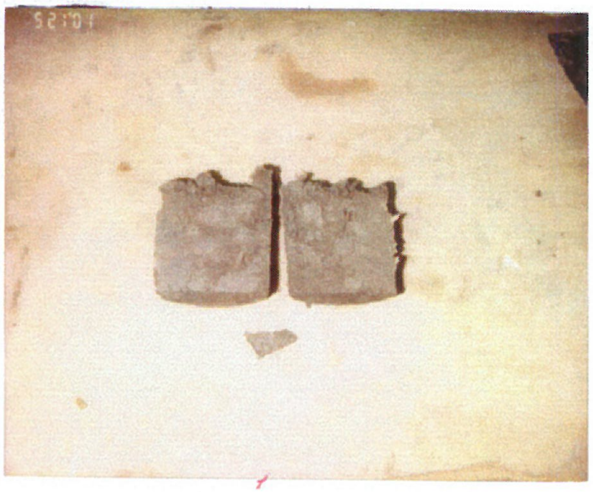
APPENDIX C-1
1697-1



EB-3, 6A at 15-18 ft.



EB-9, 5A at 12-15 ft.



EB-17, 3A at 5-8 ft.

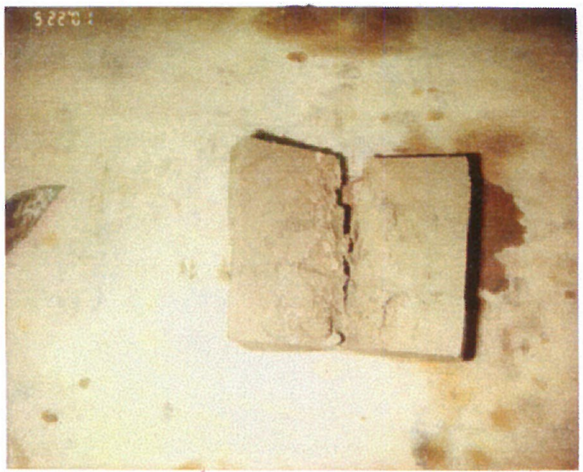
7/01*EB

HYDRAULIC CONDUCTIVITY TEST SAMPLES

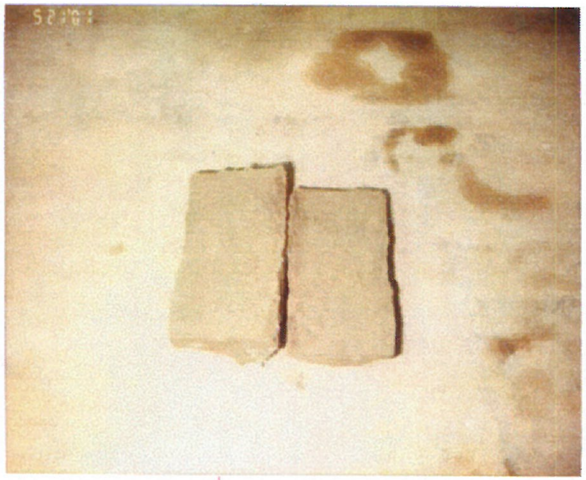
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

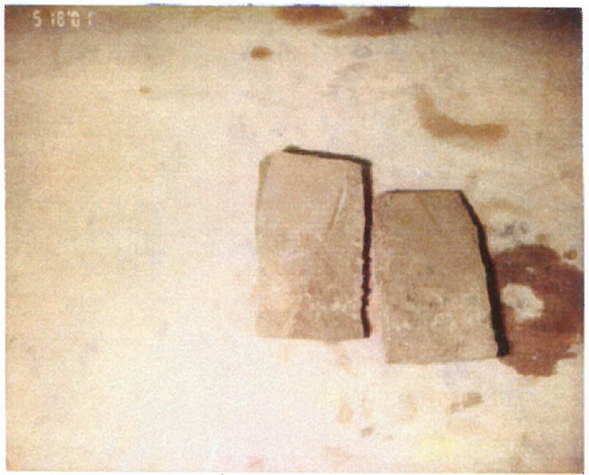
APPENDIX C-2
1697-1



EB-1, 6A at 13-16 ft. (500 psf)



EB-1, 6A at 13-16 ft.



EB-1, 6A at 13-16 ft. (4000 psf)

7/01*EB

TRIAXIAL SHEAR TEST SAMPLES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX C-3
1697-1

EB-3, 2A at 3.5-6.5 ft. (400 psf)

EB-3, 2A at 3.5-6.5 ft. (1000 psf)

EB-3, 2A at 3.5-6.5 ft. (2000 psf)

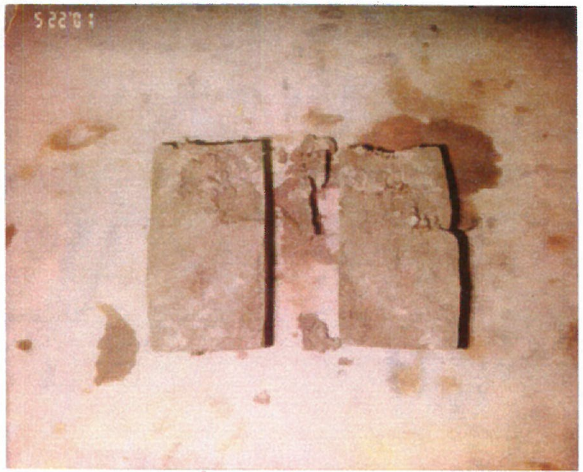
7/01*EB

TRIAXIAL SHEAR TEST SAMPLES

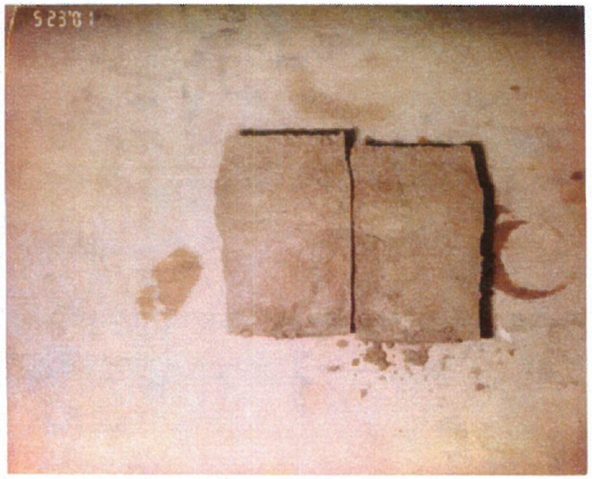
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX C-4
1697-1



EB-3, 6A at 15-18 ft. (500 psf)



EB-3, 6A at 15-18 ft. (1500 psf)

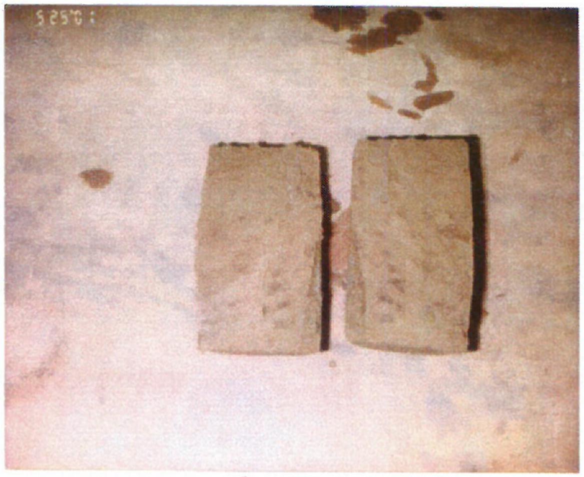
7/01*EB

TRIAXIAL SHEAR TEST SAMPLES

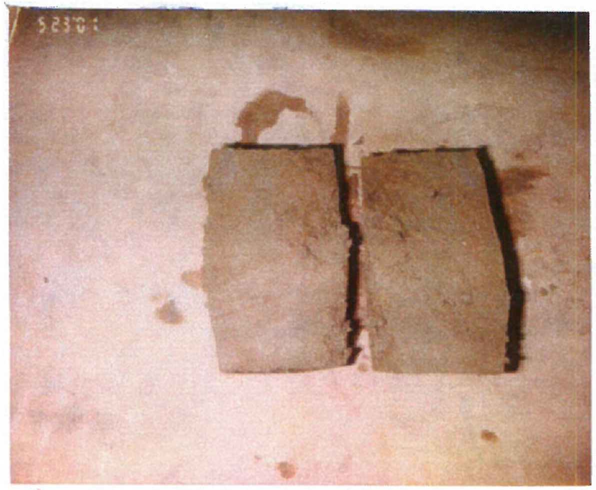
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

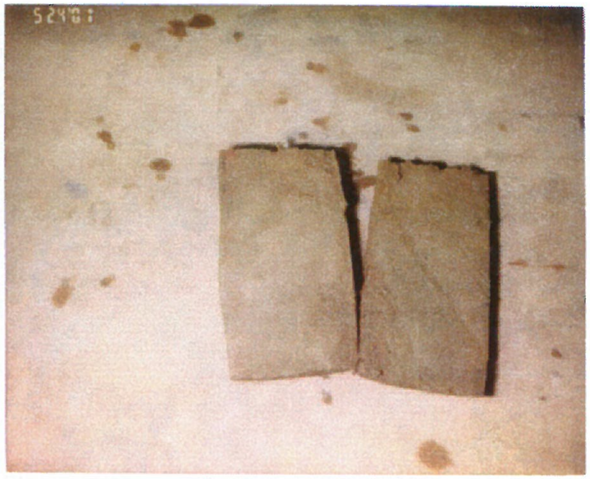
APPENDIX C-5
1697-1



EB-9, 5A at 12-15 ft. (400 psf)



EB-9, 5A at 12-15 ft. (1200 psf)



EB-9, 5A at 12-15 ft. (3000 psf)

7/01*EB

TRIAXIAL SHEAR TEST SAMPLES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX C-6
1697-1

EB-16, 3A at 5-8 ft. (400 psf)

EB-16, 3A at 5-8 ft. (1200 psf)

EB-16, 3A at 5-8 ft. (3000 psf)

7/01*EB

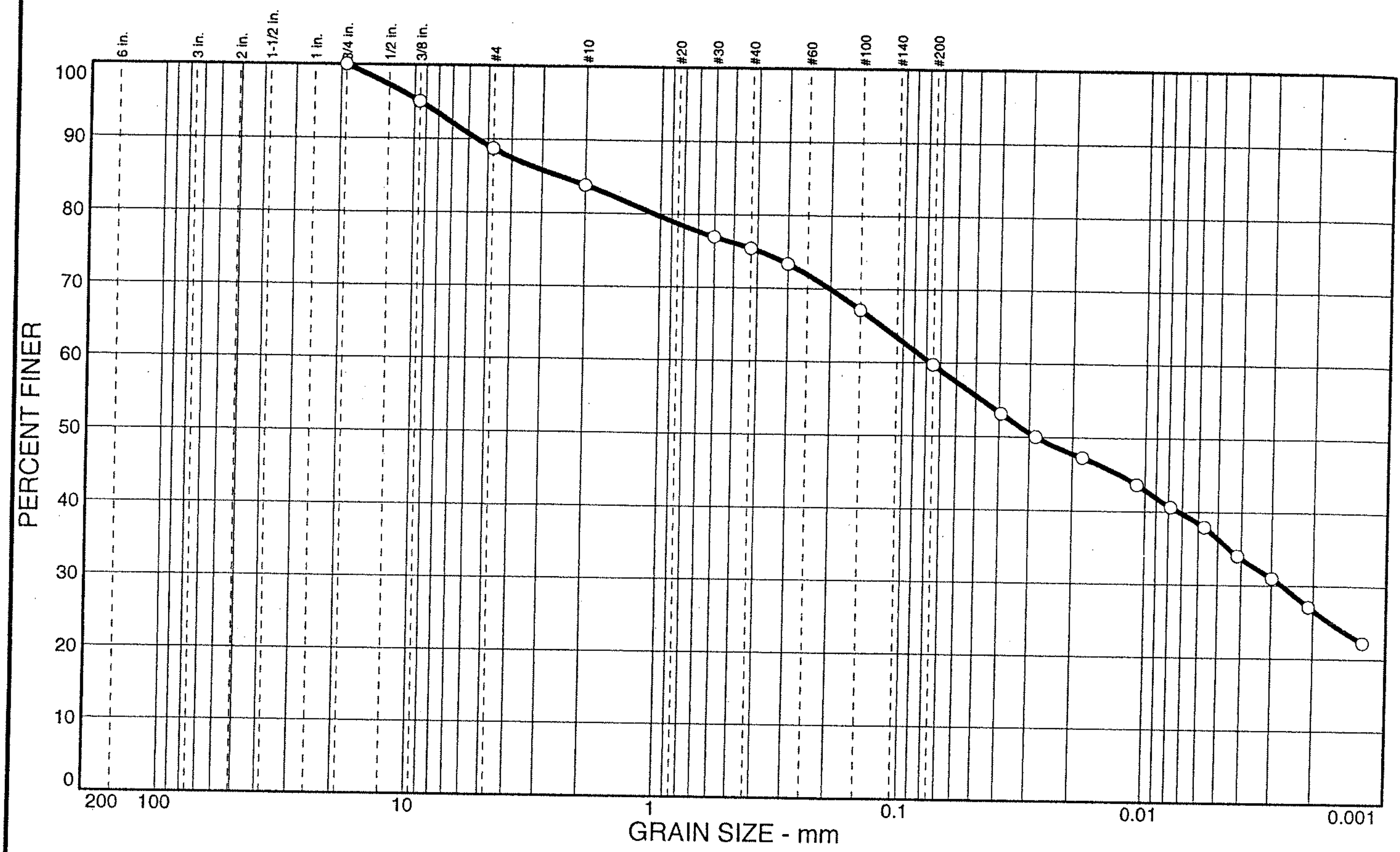
TRIAXIAL SHEAR TEST SAMPLES

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX C-7
1697-1

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○		11.3	29.0	23.3	36.4	59.7				

SIEVE inches size	PERCENT FINER		
	○		
3/4 3/8	100.0 95.0		
	GRAIN SIZE		
D_{60}	0.0772		
D_{30}	0.0027		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIZE	PERCENT FINER		
	○		
#4	88.7		
#10	83.8		
#30	76.9		
#40	75.4		
#50	73.3		
#100	67.1		
#200	59.7		
0.0392 mm.	53.1		
0.0282 mm.	50.0		
0.0181 mm.	47.2		
0.0108 mm.	43.6		
0.0078 mm.	40.6		
0.0057 mm.	37.9		
0.0041 mm.	34.0		
0.0030 mm.	31.0		
0.0021 mm.	27.1		
0.0012 mm.	22.2		

SOIL DESCRIPTION

○ gray sandy CLAY w/trace gravel

REMARKS:

○

○ Source: EB-4

Sample No.: Bucket 30

Elev./Depth: 1.5'

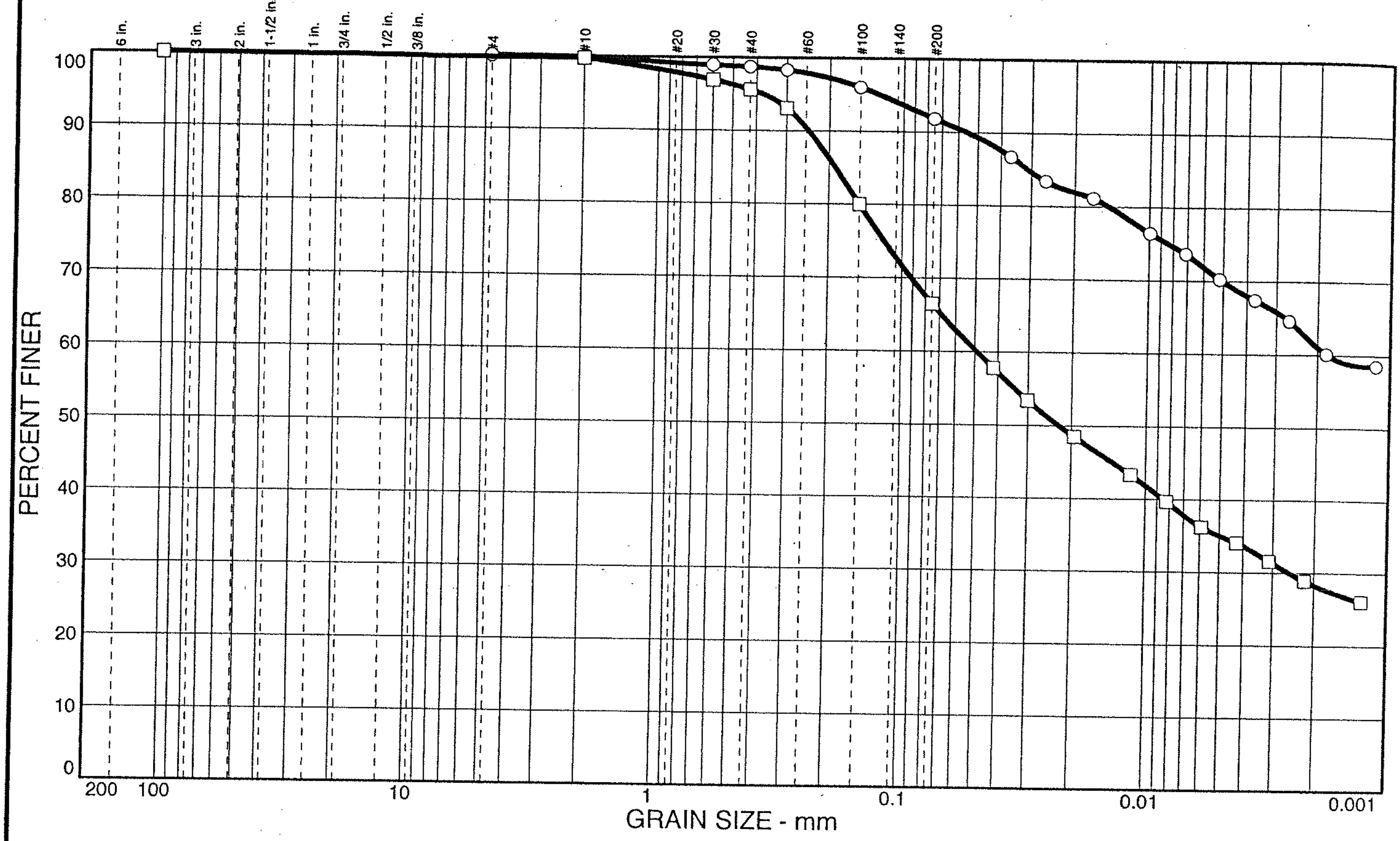
COOPER TESTING LABORATORY

Project: 1697-1

Feature:

Project No.: 028-974

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○			8.2	21.9	69.9	91.8	CH		21	72
☐		0.2	33.3	31.4	35.1	66.5	CL		14	34

SIEVE inches size	PERCENT FINER ○	PERCENT FINER ☐	
04		100.0	

	GRAIN SIZE		
D_{60}	0.0018	0.0490	
D_{30}		0.0026	
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIZE	PERCENT FINER ○	PERCENT FINER ☐	
#4	100.0		
#10	99.8	99.7	
#30	98.9	96.9	
#40	98.7	95.6	
#50	98.3	93.1	
#100	96.0	80.0	
#200	91.8	66.5	
0.0419 mm.		57.8	
0.0364 mm.	86.7		
0.0302 mm.		53.4	
0.0262 mm.	83.4		
0.0195 mm.		48.5	
0.0168 mm.	81.1		
0.0115 mm.		43.3	
0.0099 mm.	76.4		
0.0082 mm.		39.7	
0.0071 mm.	73.6		
0.0059 mm.		36.2	
0.0052 mm.	70.2		
0.0043 mm.		34.1	
0.0037 mm.	67.4		
0.0031 mm.		31.6	
0.0026 mm.	64.6		
0.0022 mm.		28.9	
0.0018 mm.	60.1		
0.0013 mm.		26.1	
0.0011 mm.	58.5		

SOIL DESCRIPTION

○ dark brown CLAY

☐ brown sandy CLAY

REMARKS:

○

☐

○ Source: EB-7　Sample No.: 2a　Elev./Depth: 3-6'

☐ Source: EB-7　Sample No.: 7a　Elev./Depth: 16-19'

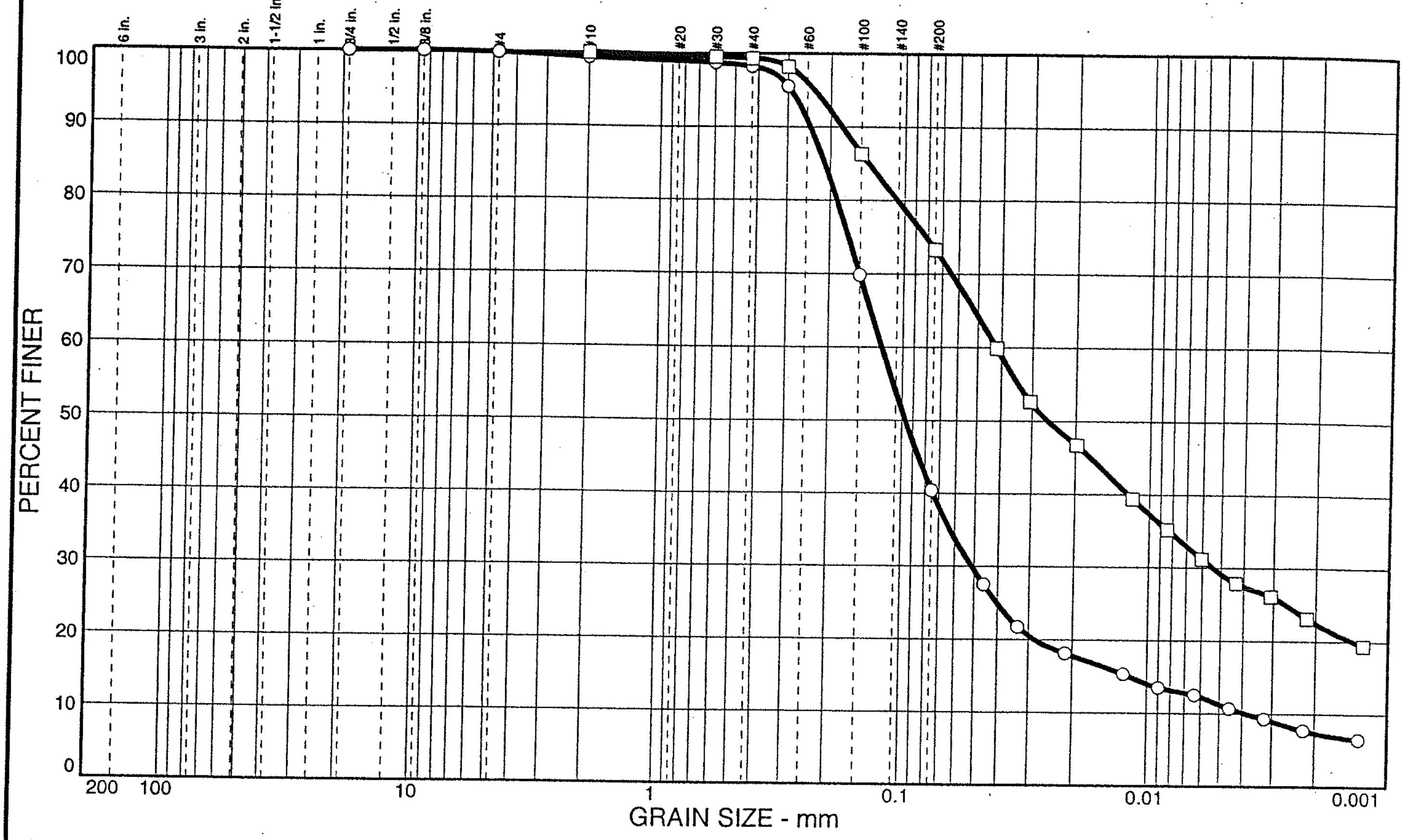
COOPER TESTING LABORATORY

Project: 1697-1

Feature:

Project No.: 028-945

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○			59.6	29.2	11.2	40.4				
☐			26.7	44.2	29.1	73.3				

SIEVE inches size	PERCENT FINER ○	PERCENT FINER ☐	
3/4	100.0		
3/8	100.0		
	GRAIN SIZE		
D_{60}	0.121	0.0420	
D_{30}	0.0512	0.0054	
D_{10}	0.0039		
	COEFFICIENTS		
C_c	5.58		
C_u	31.46		

SIZE	PERCENT FINER ○	PERCENT FINER ☐	
#4	100.0		
#10	99.3	100.0	
#30	98.7	99.5	
#40	98.2	99.3	
#50	95.5	98.2	
#100	69.8	86.3	
#200	40.4	73.3	
0.0456 mm.	27.5		
0.0419 mm.		59.9	
0.0331 mm.	21.7		
0.0305 mm.		52.6	
0.0213 mm.	18.1		
0.0198 mm.		46.7	
0.0124 mm.	15.3		
0.0117 mm.		39.4	
0.0090 mm.	13.5		
0.0084 mm.		35.2	
0.0064 mm.	12.5		
0.0061 mm.		31.2	
0.0045 mm.	10.7		
0.0043 mm.		27.9	
0.0032 mm.	9.3		
0.0031 mm.		26.1	
0.0022 mm.	7.7	23.1	
0.0013 mm.	6.5	19.4	

SOIL DESCRIPTION

○ brown silty SAND

☐ gray CLAY w/sand

REMARKS:

○

☐

○ Source: EB-8 Sample No.: 6a Elev./Depth: 15'

☐ Source: EB-8 Sample No.: 10A Elev./Depth: 35'

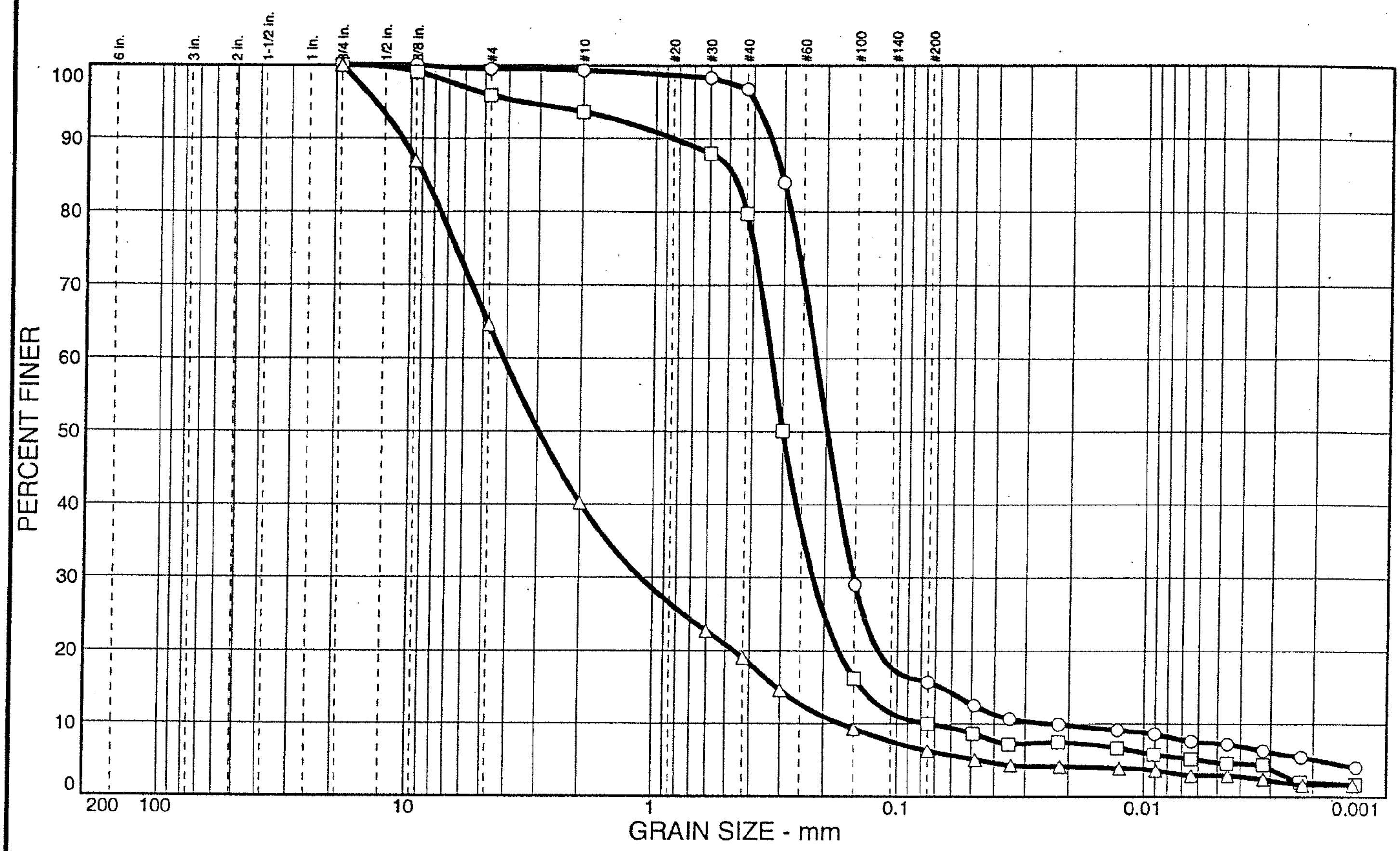
COOPER TESTING LABORATORY

Project: 1697-1

Feature:

Project No.: 028-945

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○		0.5	83.8	8.4	7.3	15.7				
□		4.1	85.9	5.3	4.7	10.0				
△		35.4	58.3	3.4	2.9	6.3	SW-SM			

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
3/4	100.0	100.0	100.0
3/8	100.0	99.1	87.0
	GRAIN SIZE		
D$_{60}$	0.221	0.334	4.13
D$_{30}$	0.153	0.223	1.12
D$_{10}$	0.0239	0.0750	0.170
	COEFFICIENTS		
C$_{c}$	4.42	1.99	1.78
C$_{u}$	9.25	4.46	24.24

SIZE	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	99.5	95.9	64.6
#10	99.3	93.6	40.2
#30	98.3	87.9	22.7
#40	96.7	79.7	19.0
#50	83.9	50.1	14.6
#100	29.0	16.2	9.3
#200	15.7	10.0	6.3
0.0491 mm.		8.7	
0.0485 mm.	12.5		
0.0479 mm.			5.1
0.0349 mm.		7.2	
0.0346 mm.	10.7		
0.0342 mm.			4.3
0.0221 mm.		7.5	
0.0220 mm.	9.9		
0.0217 mm.			4.1
0.0128 mm.		6.7	
0.0127 mm.	9.1		
0.0125 mm.			3.9
0.0091 mm.		5.8	
0.0090 mm.	8.6		
0.0089 mm.			3.6
0.0064 mm.	7.6	5.2	2.9
0.0046 mm.		4.6	
0.0045 mm.	7.2		2.9
0.0032 mm.	6.3	4.4	2.4
0.0023 mm.		2.0	
0.0022 mm.	5.4		1.6
0.0014 mm.			1.6
0.0013 mm.	4.0	1.7	

SOIL DESCRIPTION

○ gray silty SAND

□ gray SAND w/silt

△ gray well-graded Sand with silt and gravel

REMARKS:

○

□

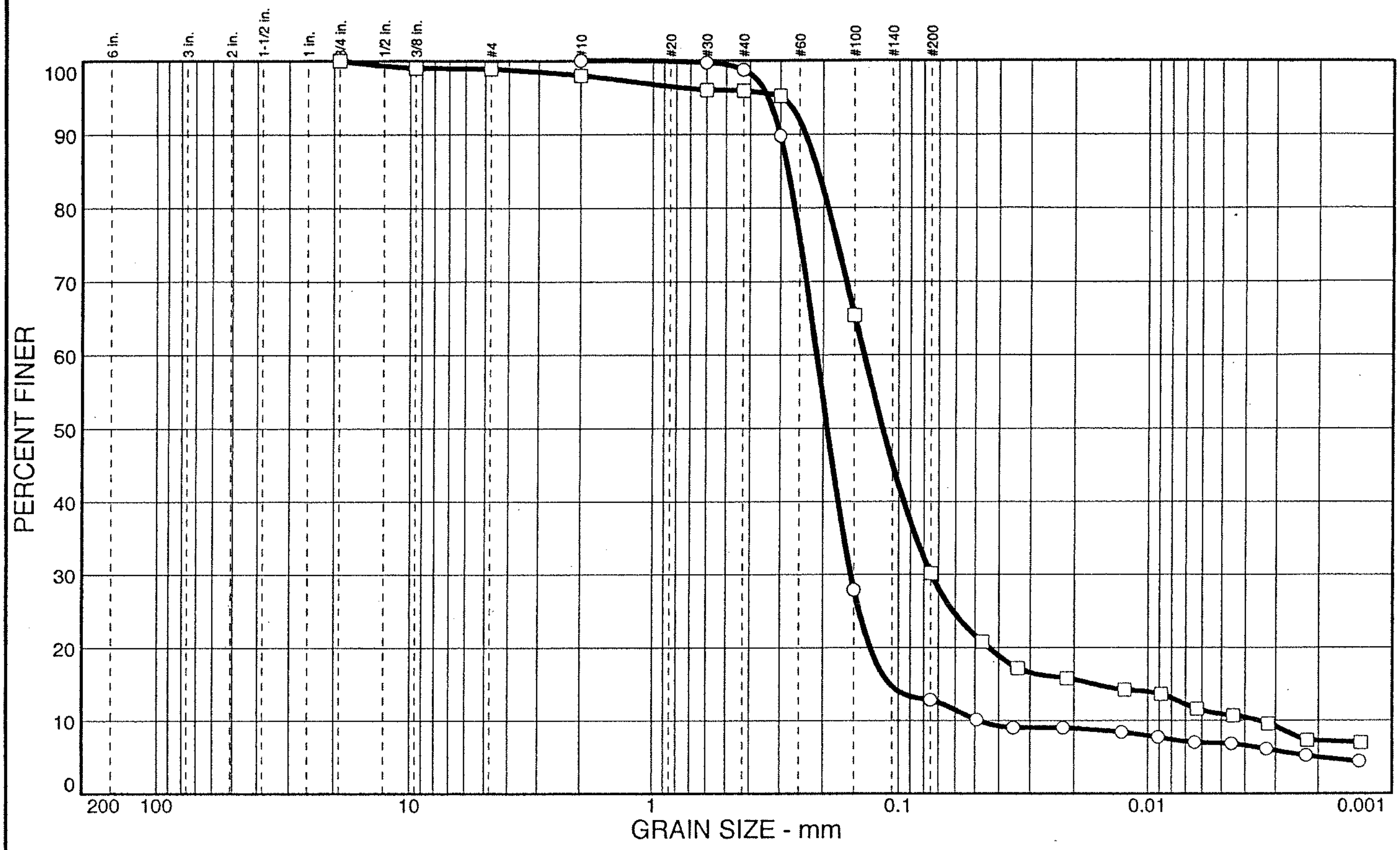
△

○ Source: EB-10	Sample No.: 6a	Elev./Depth: 15.5'
□ Source: EB-10	Sample No.: 8A	Elev./Depth: 26'
△ Source: EB-10	Sample No.: 9a	Elev./Depth: 31'

COOPER TESTING LABORATORY

Project: 1697-1
Feature:
Project No.: 028-945

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○			87.2	5.9	6.9	12.8	SM			
□		1.1	68.7	19.3	10.9	30.2	SM			-

SIEVE inches size	PERCENT FINER		
	○	□	
3/4		100.0	
3/8		99.0	
	GRAIN SIZE		
D_{60}	0.212	0.137	
D_{30}	0.154	0.0745	
D_{10}	0.0482	0.0035	
	COEFFICIENTS		
C_c	2.33	11.60	
C_u	4.40	39.37	

SIZE	PERCENT FINER		
	○	□	
#4		98.9	
#10	100.0	98.0	
#30	99.7	96.0	
#40	98.7	95.9	
#50	89.8	95.2	
#100	27.9	65.4	
#200	12.8	30.2	
0.0491 mm.	10.1		
0.0468 mm.		20.8	
0.0349 mm.	9.0		
0.0336 mm.		17.2	
0.0221 mm.	9.0		
0.0214 mm.		15.8	
0.0128 mm.	8.4		
0.0124 mm.		14.2	
0.0091 mm.	7.7		
0.0088 mm.		13.6	
0.0064 mm.	7.0		
0.0063 mm.		11.6	
0.0045 mm.	6.8	10.7	
0.0033 mm.	6.1		
0.0032 mm.		9.6	
0.0022 mm.	5.2	7.3	
0.0014 mm.	4.4		
0.0013 mm.		7.0	

SOIL DESCRIPTION

○ grayish brown silty SAND

□ grayish brown silty SAND

REMARKS:

○

□

○ Source: EB-11 Elev./Depth: 30.0’

□ Source: EB-12 Elev./Depth: 25.5’

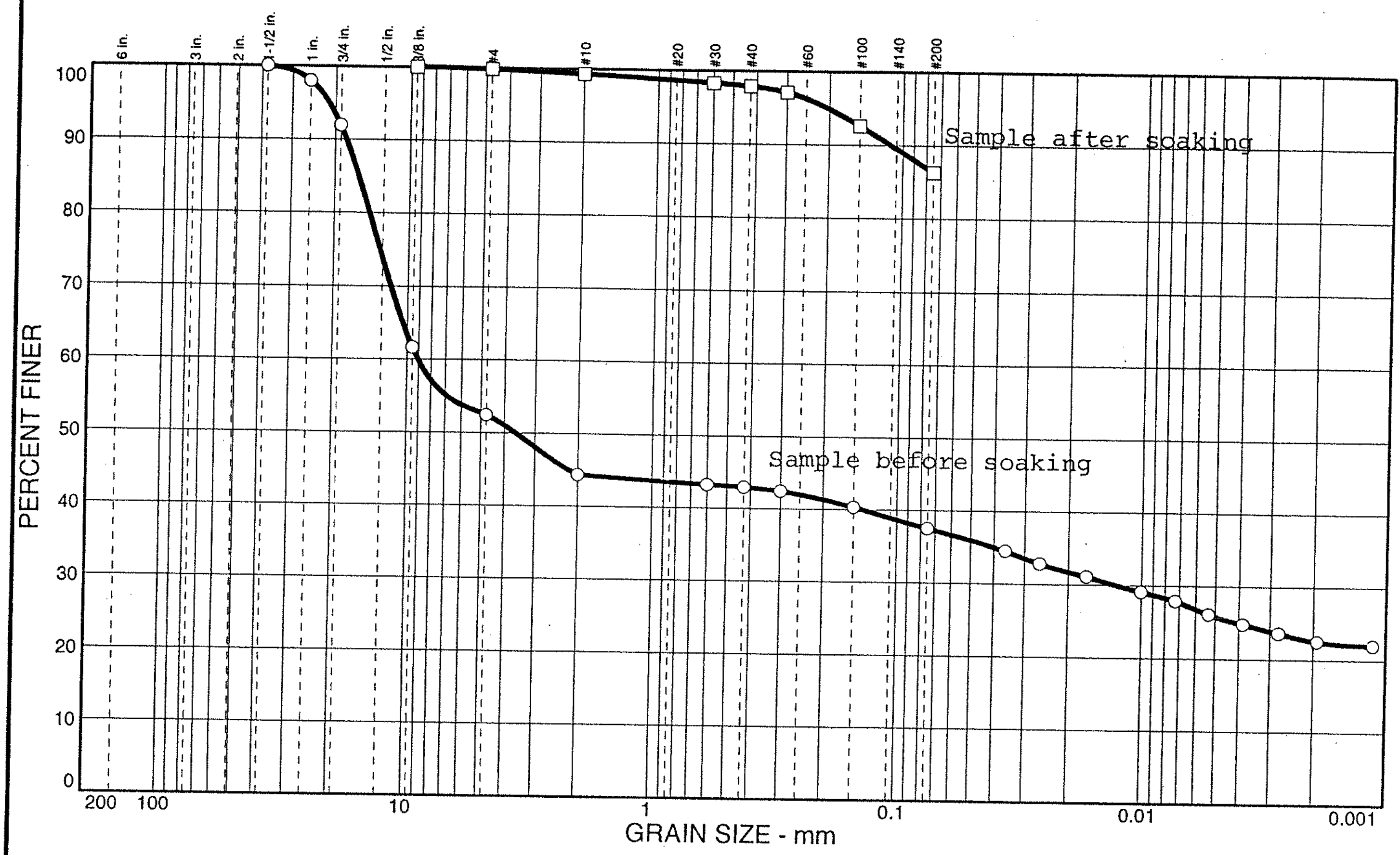
COOPER TESTING LABORATORY

Project: 1697-1 / 108GI /LP

Feature:

Project No.: 028-957

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○		47.6	14.9	11.7	25.8	37.5	GM			
☐		0.2	13.5			86.3				

SIEVE inches size	PERCENT FINER ○	PERCENT FINER ☐	
1.5	100.0		
1	98.0		
3/4	91.9		
3/8	61.6	100.0	
	GRAIN SIZE		
D_{60}	9.04		
D_{30}	0.0128		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIZE	PERCENT FINER ○	PERCENT FINER ☐	
#4	52.4	99.8	
#10	44.3	99.3	
#30	43.2	98.3	
#40	42.9	97.9	
#50	42.4	97.1	
#100	40.3	92.6	
#200	37.5	86.3	
0.0360 mm.	34.5		
0.0260 mm.	32.8		
0.0168 mm.	31.1		
0.0101 mm.	29.1		
0.0073 mm.	27.9		
0.0053 mm.	26.1		
0.0038 mm.	24.8		
0.0028 mm.	23.6		
0.0019 mm.	22.5		
0.0011 mm.	22.0		

SOIL DESCRIPTION

○ black clayey GRAVEL w/sand (see note)

☐ black CLAY (weathered Shale)

REMARKS:

○ The gravel fell apart when soaked in water. It apears to be a weathered shale. It is very hard when dry but immediately breaks down to soil

☐ This is the gravel sample after soaking over night.

○ Source: EB-15
☐ Source: EB-15

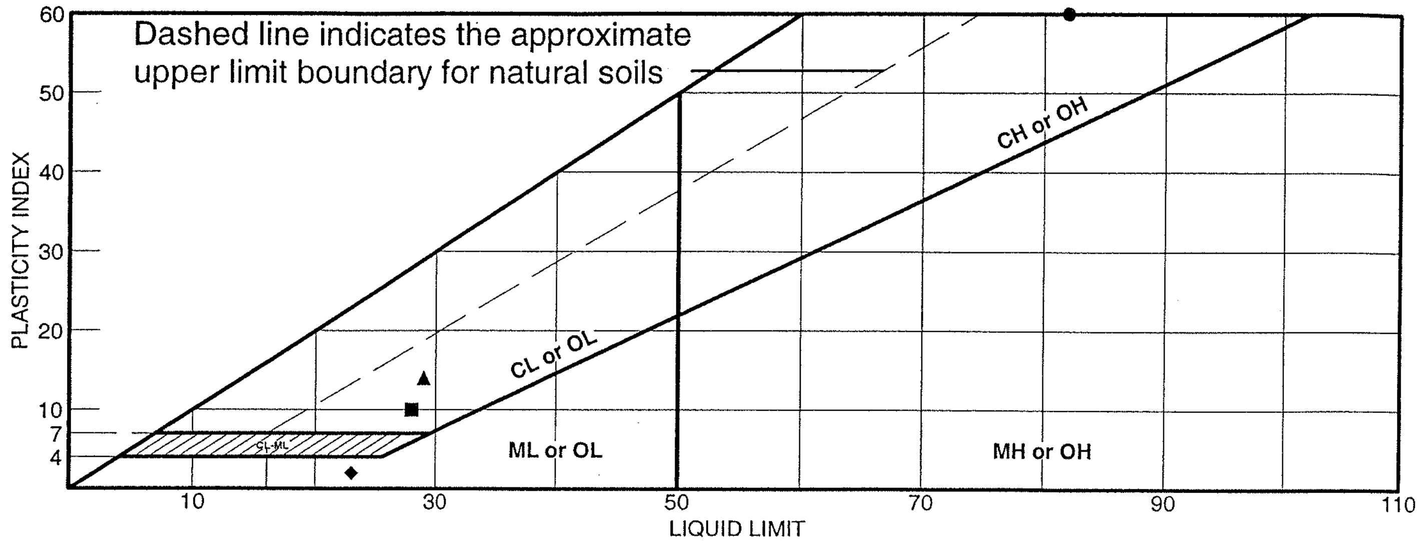
Sample No.: 22 Soaked

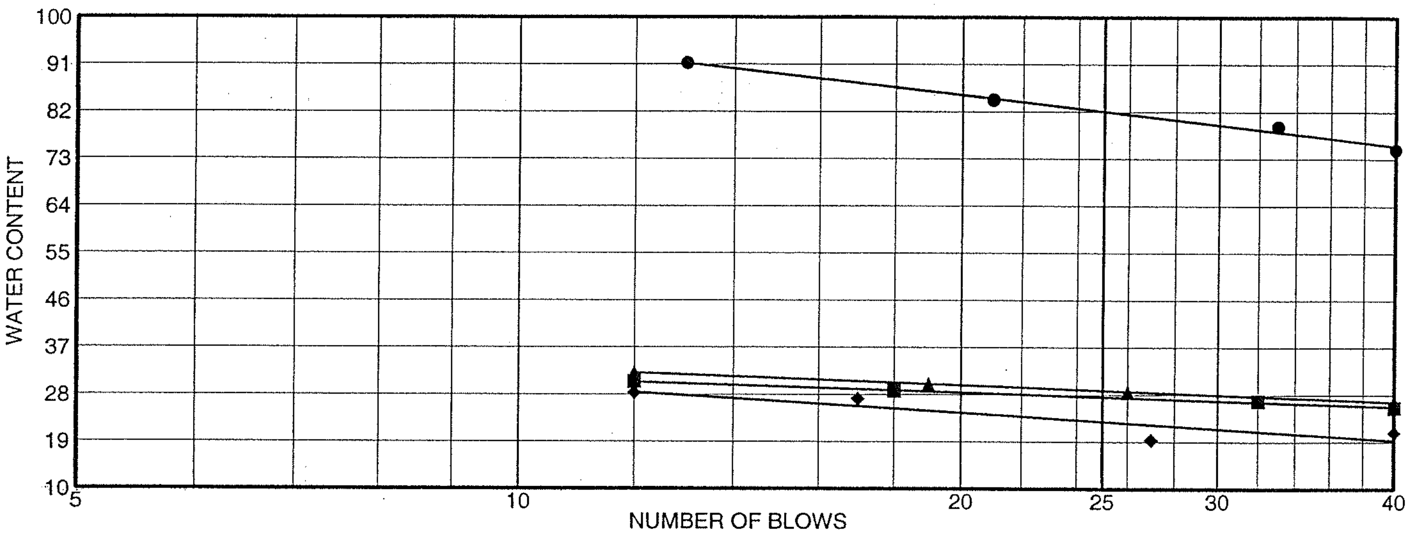
Elev./Depth: 4.0'
Elev./Depth: 4.0'

COOPER TESTING LABORATORY

Project: 1697-1
Feature:
Project No.: 028-974

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	light brown CLAY with trace sand	82	22	60			
■	brown clayey SAND	28	18	10			
▲	brown sandy CLAY (possibly clay with sand)	29	15	14			
◆	brown silty SAND / sandy silt	23	21	2			

Project No. 028-957 **Client:** Lowney

Project: 1697-1 / 108GI /LP

● **Source:** EB-1 **Sample No.:** 6A **Elev./Depth:** 13-16
■ **Source:** EB-2 **Sample No.:** 2A **Elev./Depth:** 3-6
▲ **Source:** EB-3 **Sample No.:** 6A **Elev./Depth:** 15-18'
◆ **Source:** EB-9 **Sample No.:** 5A **Elev./Depth:** 12-15

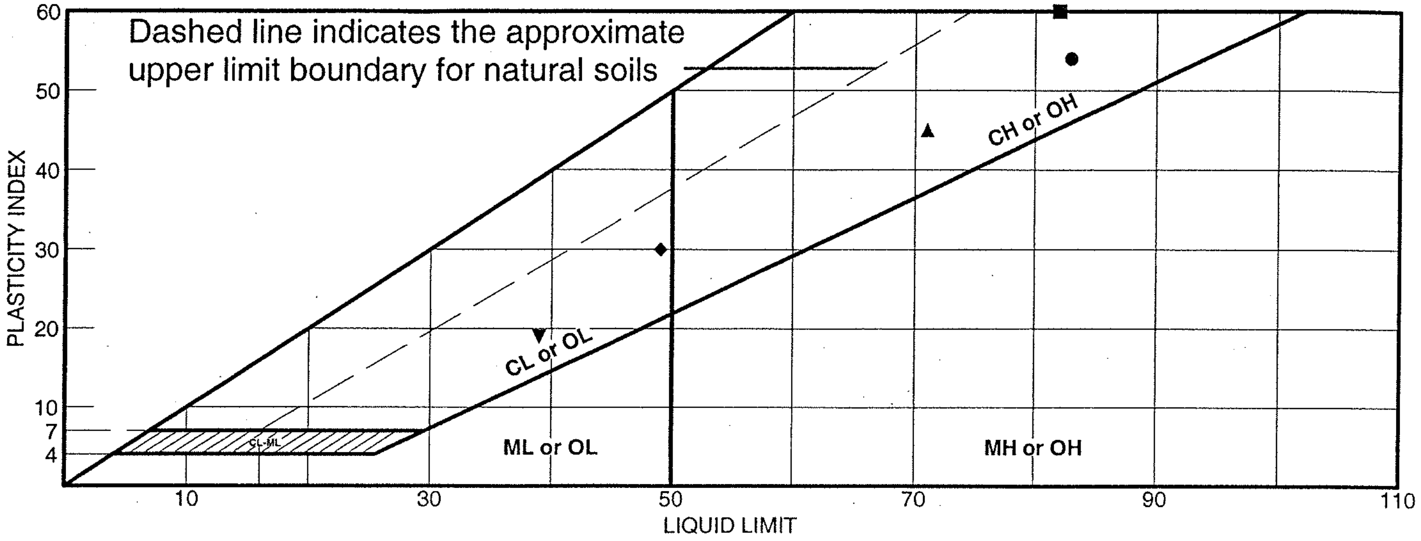
Remarks:
●
■
▲
◆

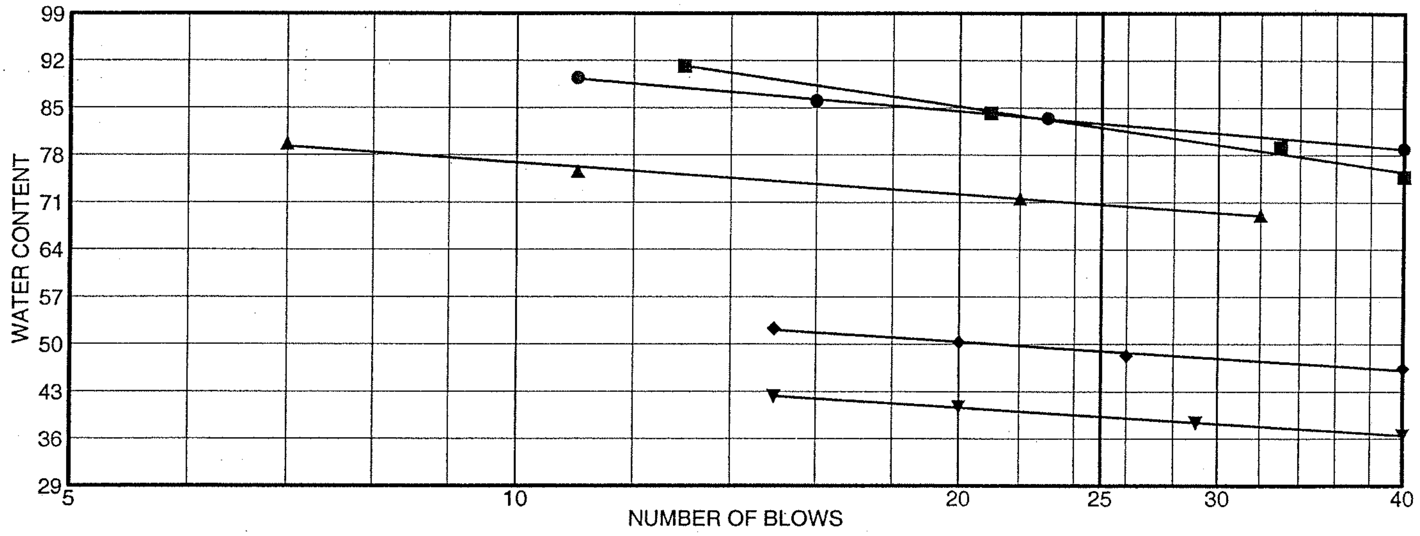
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	black CLAY	83	29	54			
■	light brown CLAY with trace sand	82	22	60			
▲	black CLAY	71	26	45			
◆	light gray CLAY, friable, crumbly	49	19	30			
▼	brown CLAY with sand	39	20	19			

Project No. 028-957 **Client:** Lowney

Project: 1697-1 / 108GI /LP

● **Source:** EB-1	**Sample No.:** 4A	**Elev./Depth:** 7-10’
■ **Source:** EB-1	**Sample No.:** 6A	**Elev./Depth:** 13-16
▲ **Source:** EB-3	**Sample No.:** 2A	**Elev./Depth:** 3.5-6.5’
◆ **Source:** EB-3	**Sample No.:** 4A	**Elev./Depth:** 9-12’
▼ **Source:** EB-17	**Sample No.:** 3A	**Elev./Depth:** 5.8

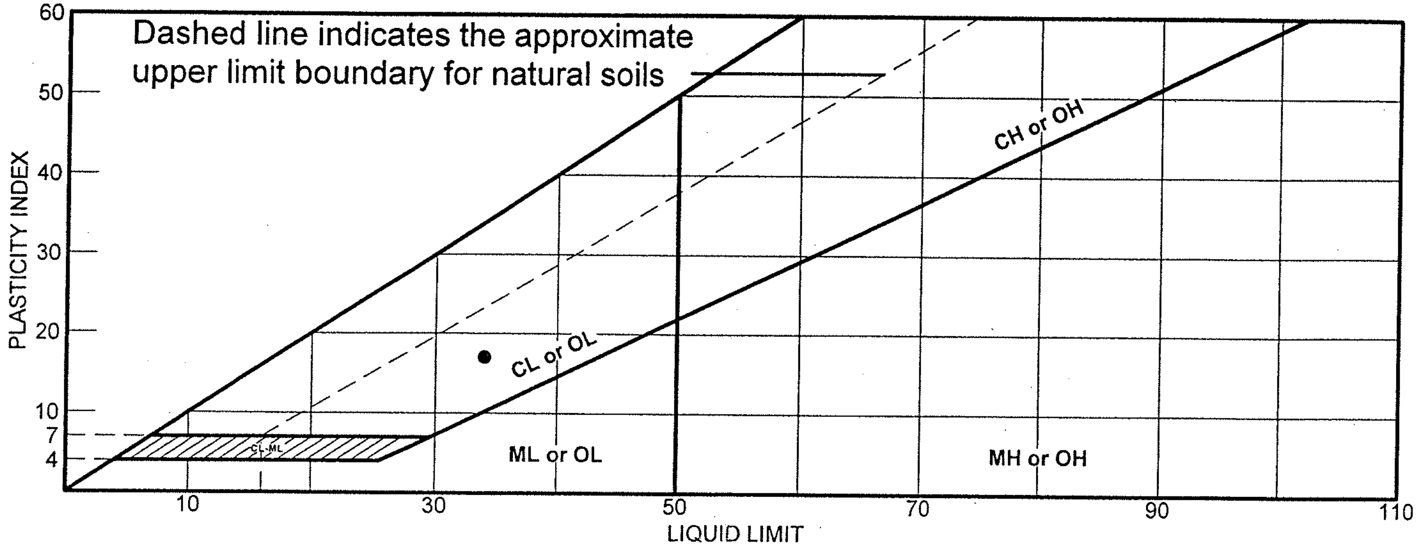
Remarks:
●
■
▲
◆
▼

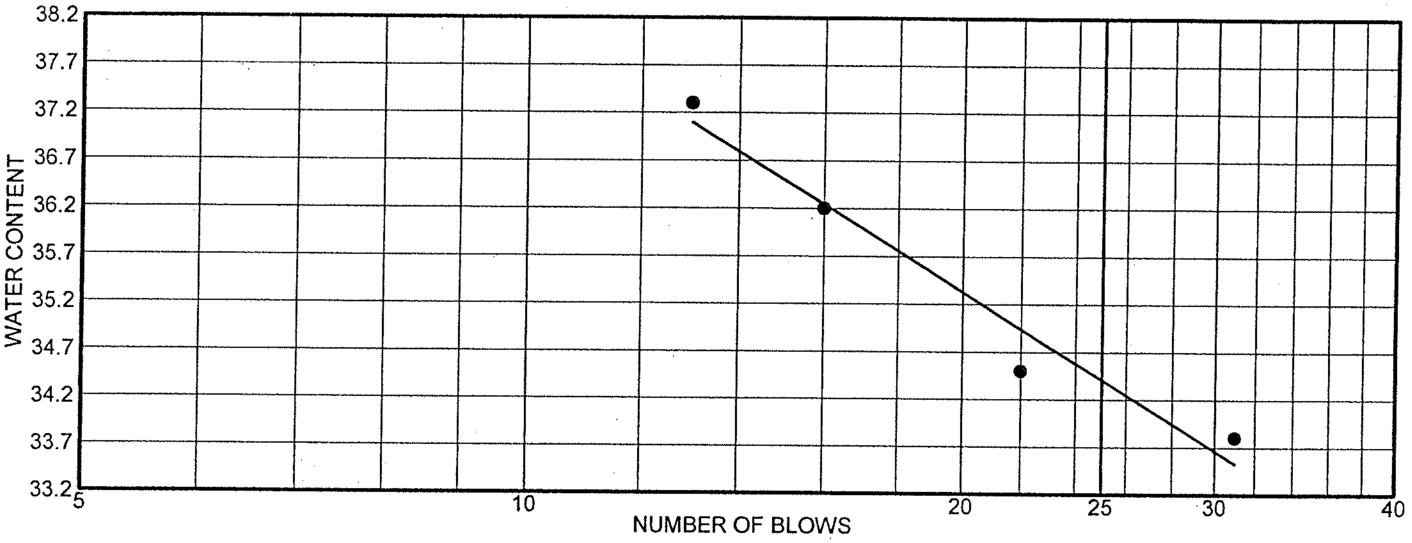
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	gray sandy CLAY w/trace gravel	34	17	17	75.4	59.7	

Project No. 028-974 **Client:** Lowney

Project: 1697-1

● **Source:** EB-4 **Sample No.:** Bucket 30 **Elev./Depth:** 1.5'

Remarks:

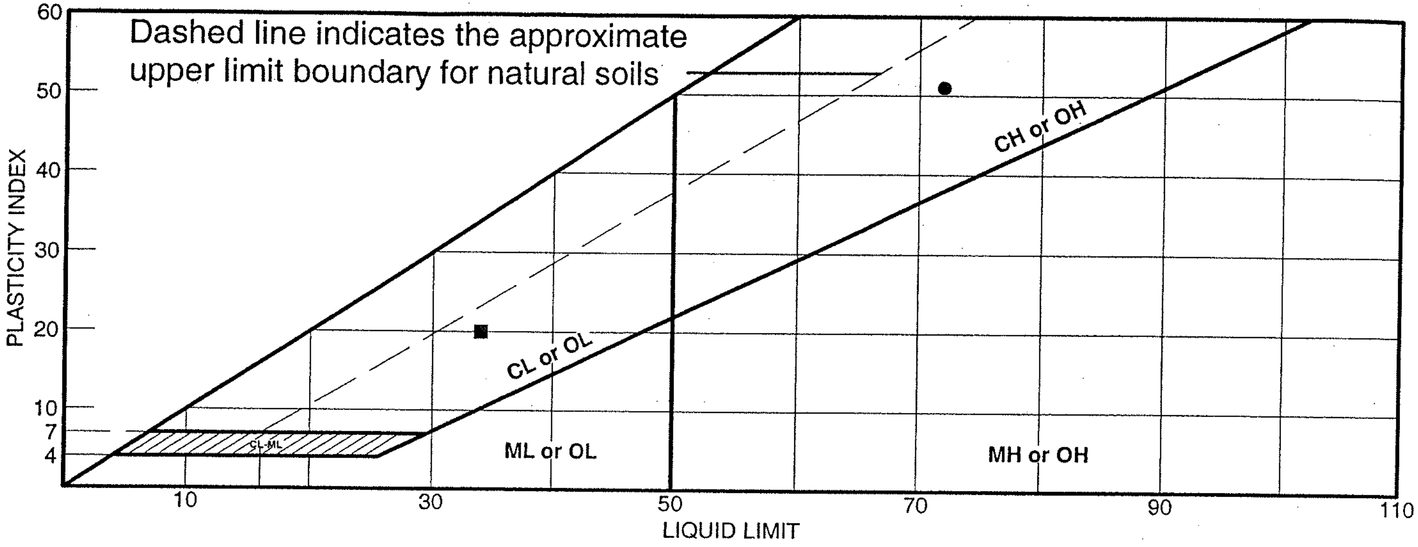
●

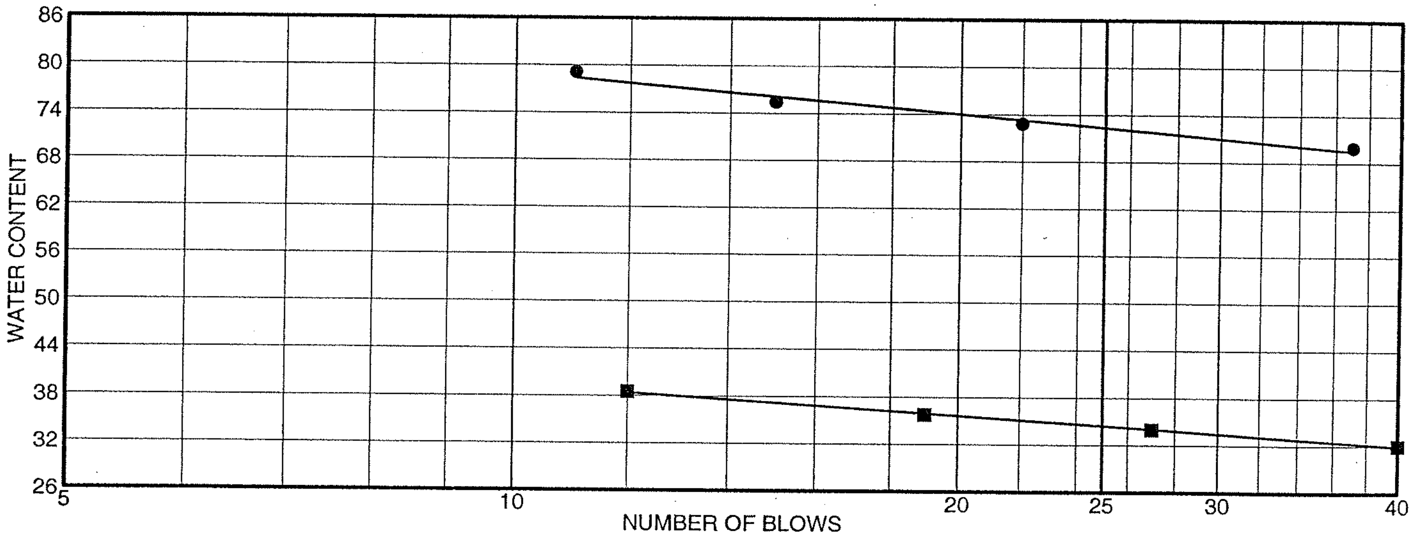
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	dark brown CLAY	72	21	51	98.7	91.8	CH
■	brown sandy CLAY	34	14	20	95.6	66.5	CL

Project No. 028-945 **Client:** Lowney

Project: 1697-1

● **Source:** EB-7 **Sample No.:** 2a **Elev./Depth:** 3-6’

■ **Source:** EB-7 **Sample No.:** 7a **Elev./Depth:** 16-19’

Remarks:

●

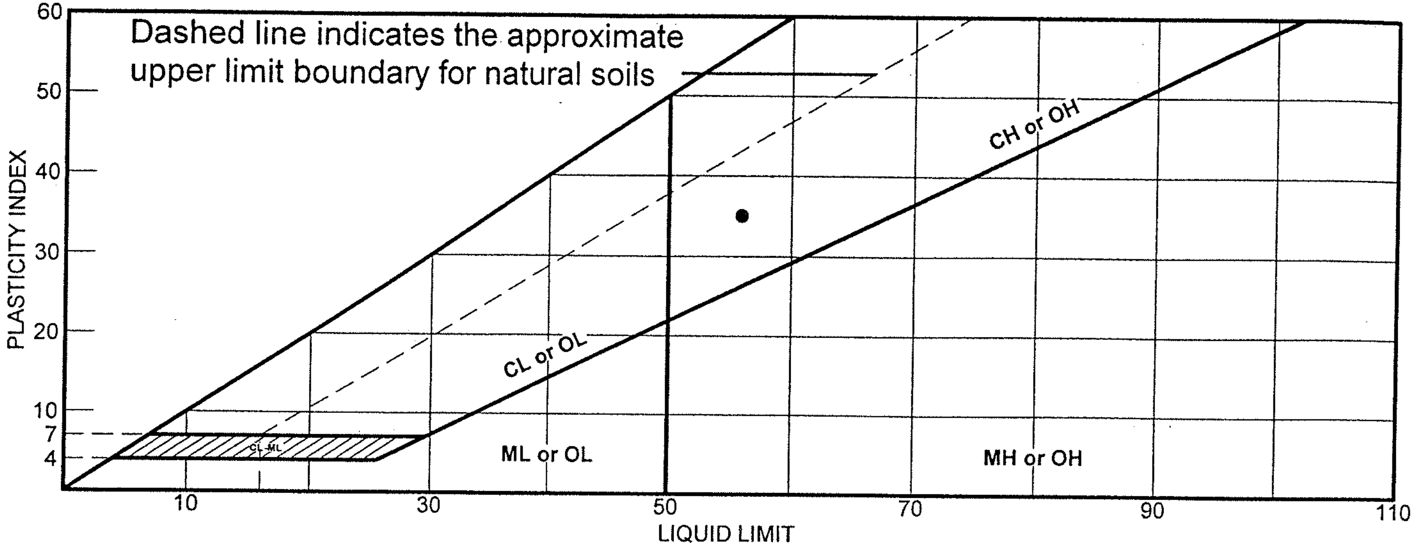
■

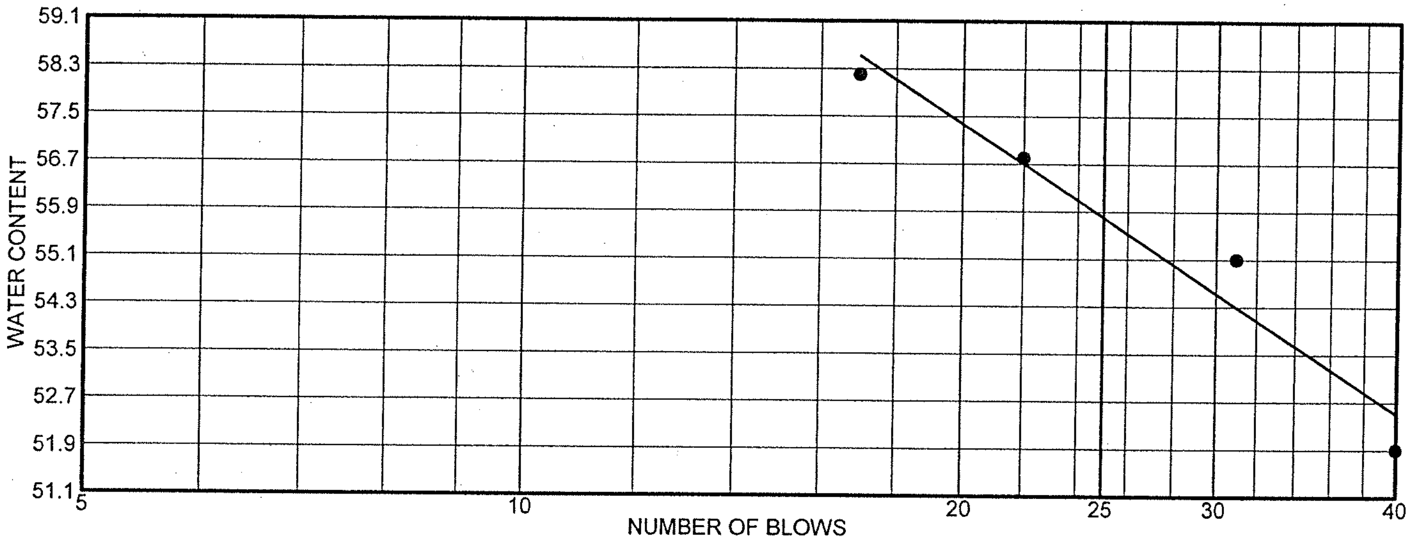
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	black fat CLAY	56	21	35	97.9	86.3	CH

Project No. 028-974 **Client:** Lowney

Project: 1697-1

● **Source:** EB-15 **Sample No.:** 22 Soaked **Elev./Depth:** 4.0'

Remarks:

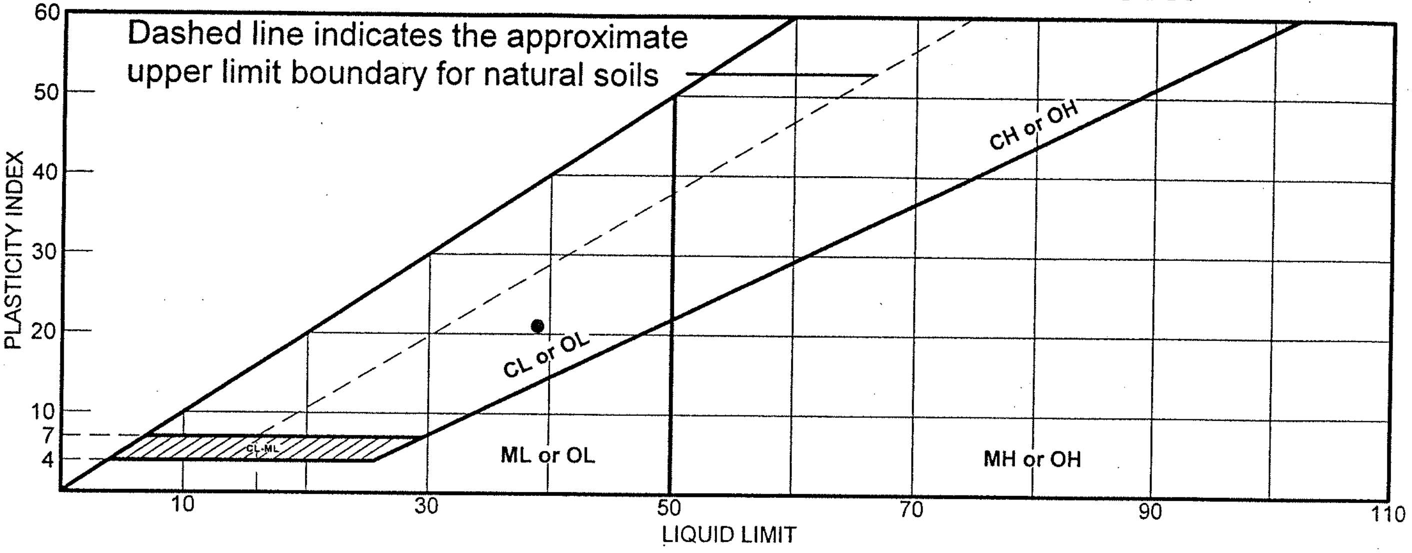
●

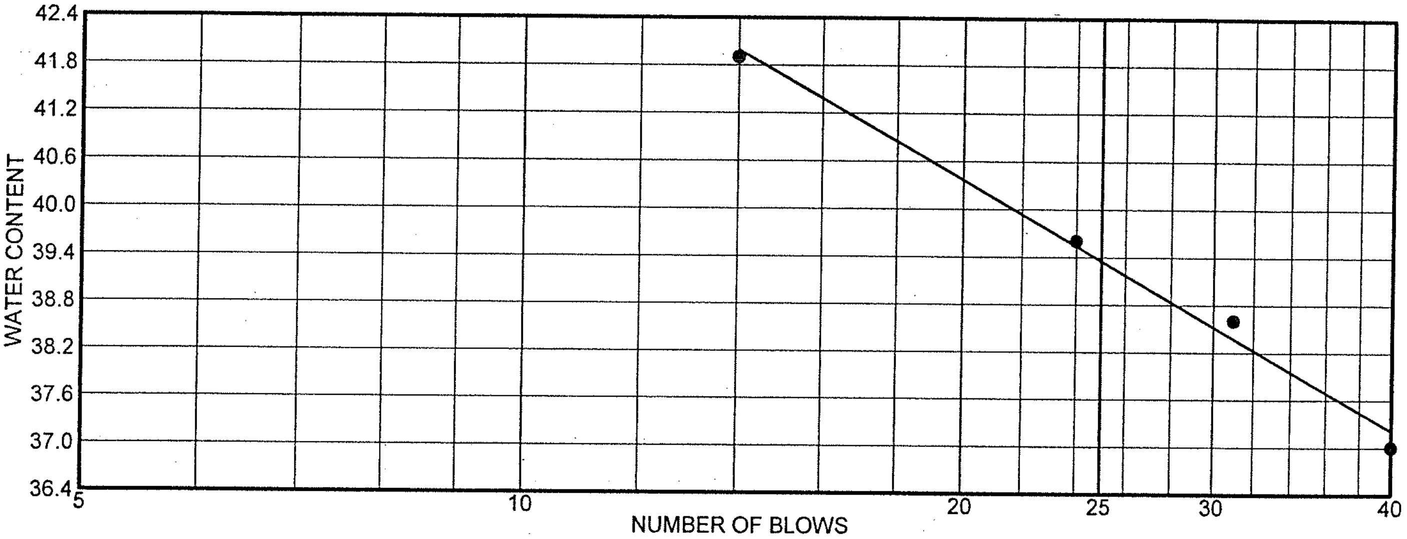
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	brown CLAY with sand and gravel	39	18	21			

Project No. 028-974 **Client:** Lowney

Project: 1697-1 / Berryessa

● **Source:** EB-15 **Sample No.:** bucket 5

Remarks:

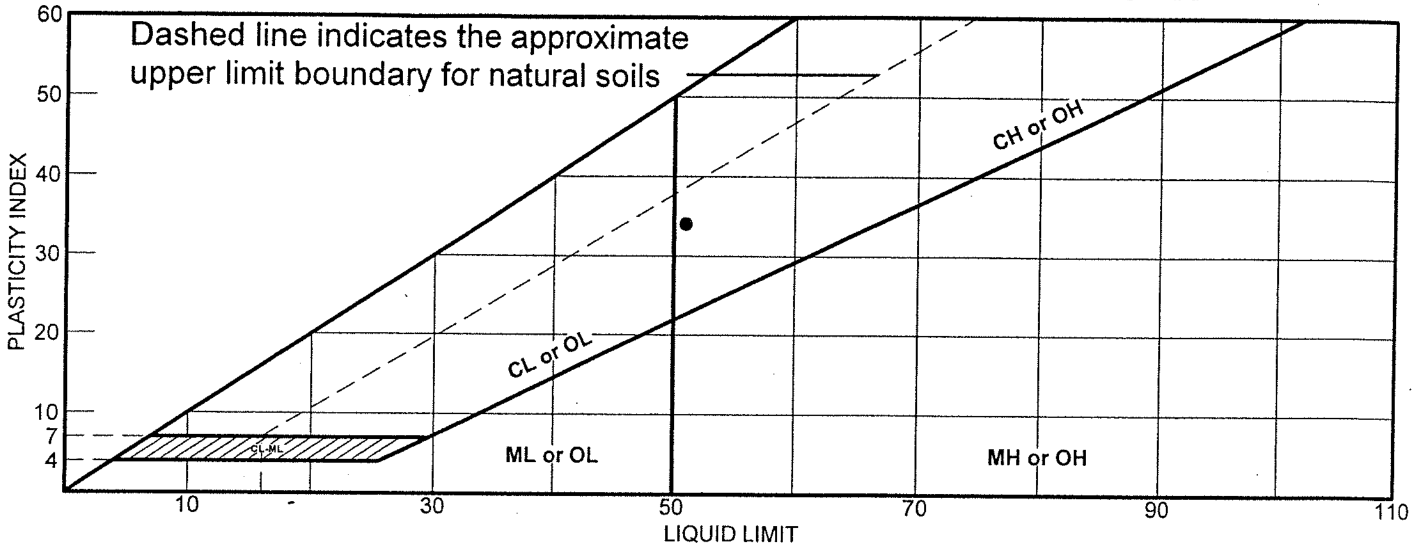
●

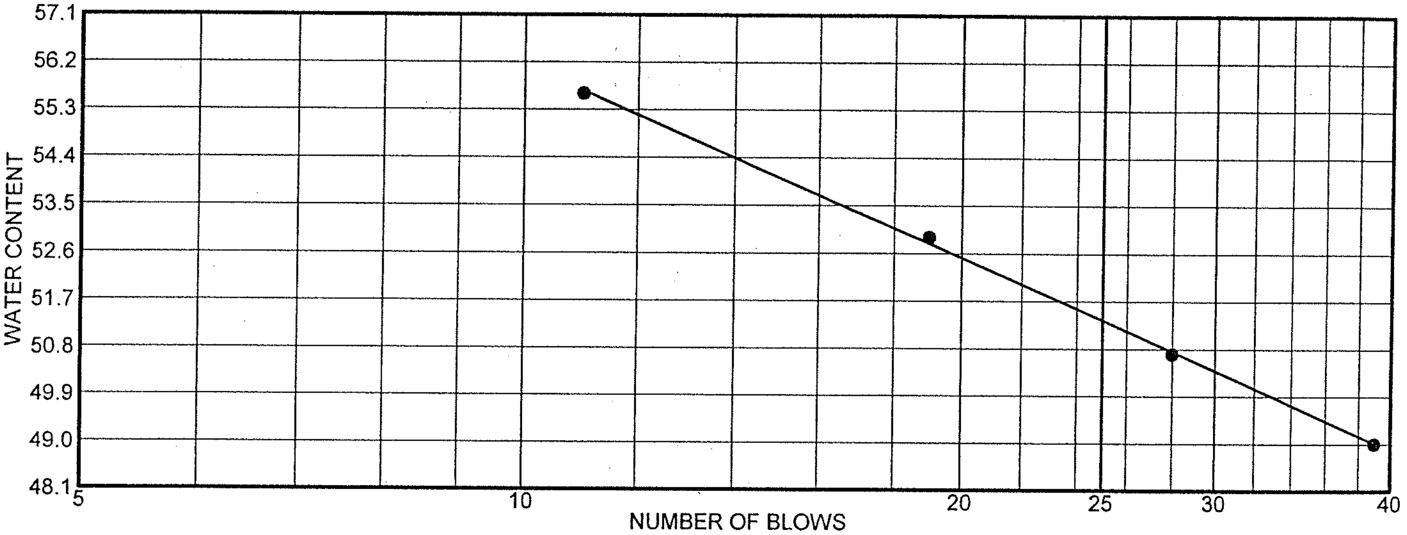
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	brown mottled gray sandy CLAY	51	17	34			

Project No. 028-975 **Client:** Lowney

Project: 1697-1 / Berryessa

● **Source:** EB15 **Sample No.:** 4A **Elev./Depth:** 11-14

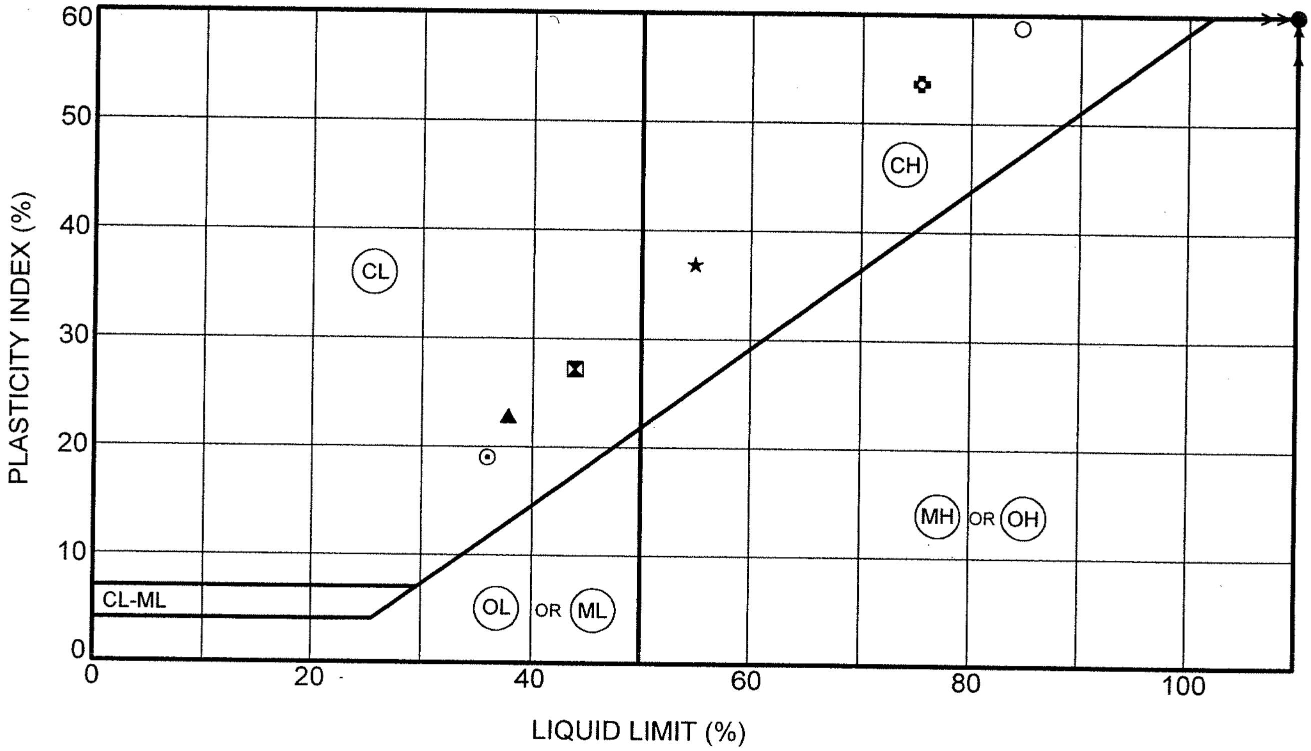
Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate



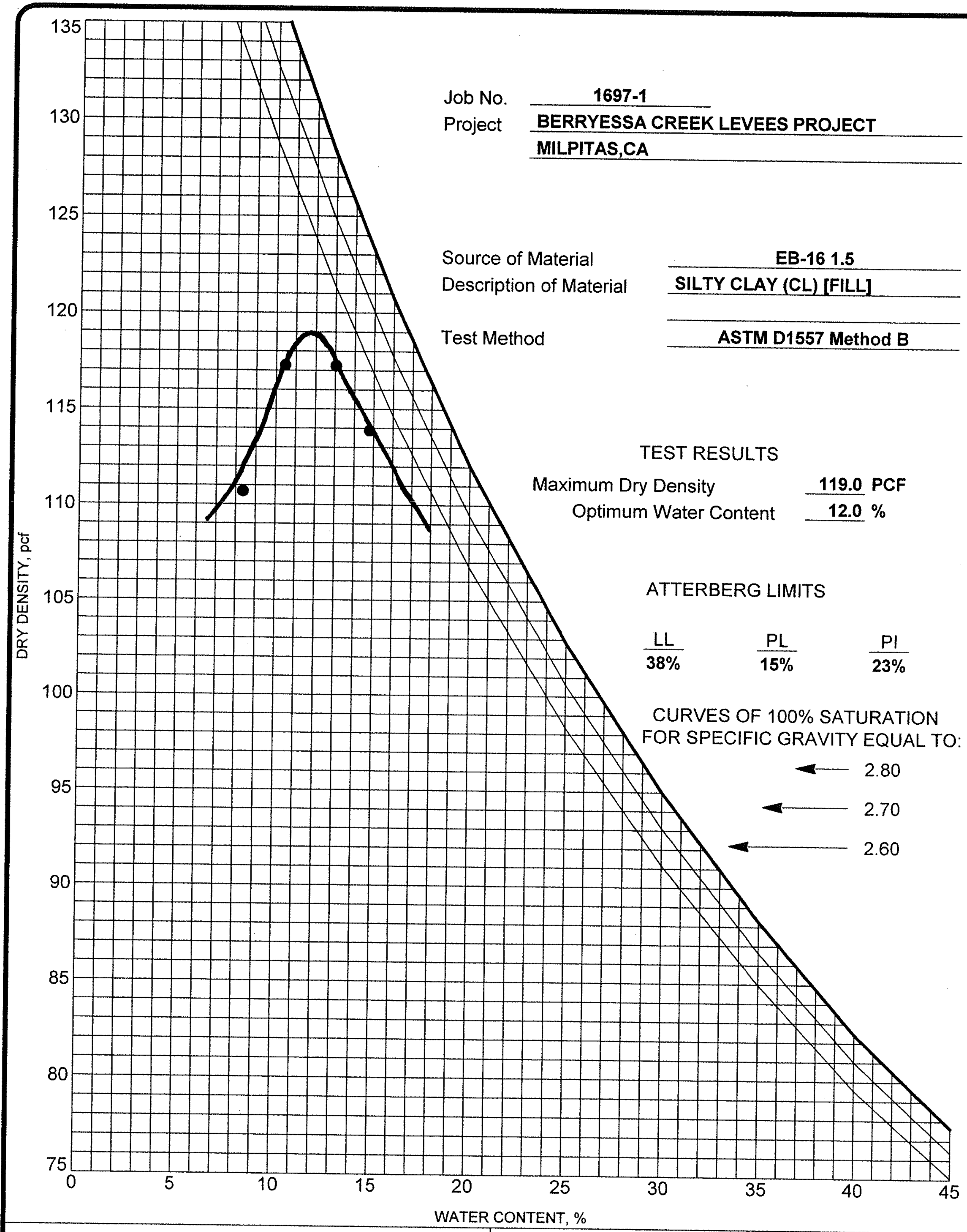
Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-10**	**4.5**	**33**	**124**	**21**	**103**		**SILTY CLAY (CH)**
☒	**EB-14**	**2.0**	**17**	**44**	**17**	**27**		**SILTY CLAY (CL) [FILL]**
▲	**EB-16**	**1.5**	**14**	**38**	**15**	**23**		**SILTY CLAY (CL) [FILL]**
★	**EB-16**	**7.5**	**23**	**55**	**18**	**37**		**SILTY CLAY (CH)**
⊙	**EB-2**	**2.0**	**19**	**36**	**17**	**19**		**SILTY CLAY (CL)**
✿	**EB-7**	**7.5**	**31**	**75**	**22**	**53**		**SILTY CLAY (CH)**
○	**EB-9**	**1.5**	**30**	**85**	**26**	**59**		**SILTY CLAY (CH)**

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

PLASTICITY CHART AND DATA

Project: BERRYESSA CREEK LEVEES PROJECT
Location: MILPITAS,CA
Project No.: 1697-1

FIGURE B-1



Job No. **1697-1**
Project **BERRYESSA CREEK LEVEES PROJECT**
MILPITAS,CA

Source of Material **EB-16 1.5**
Description of Material **SILTY CLAY (CL) [FILL]**

Test Method **ASTM D1557 Method B**

TEST RESULTS

Maximum Dry Density **119.0** PCF
Optimum Water Content **12.0** %

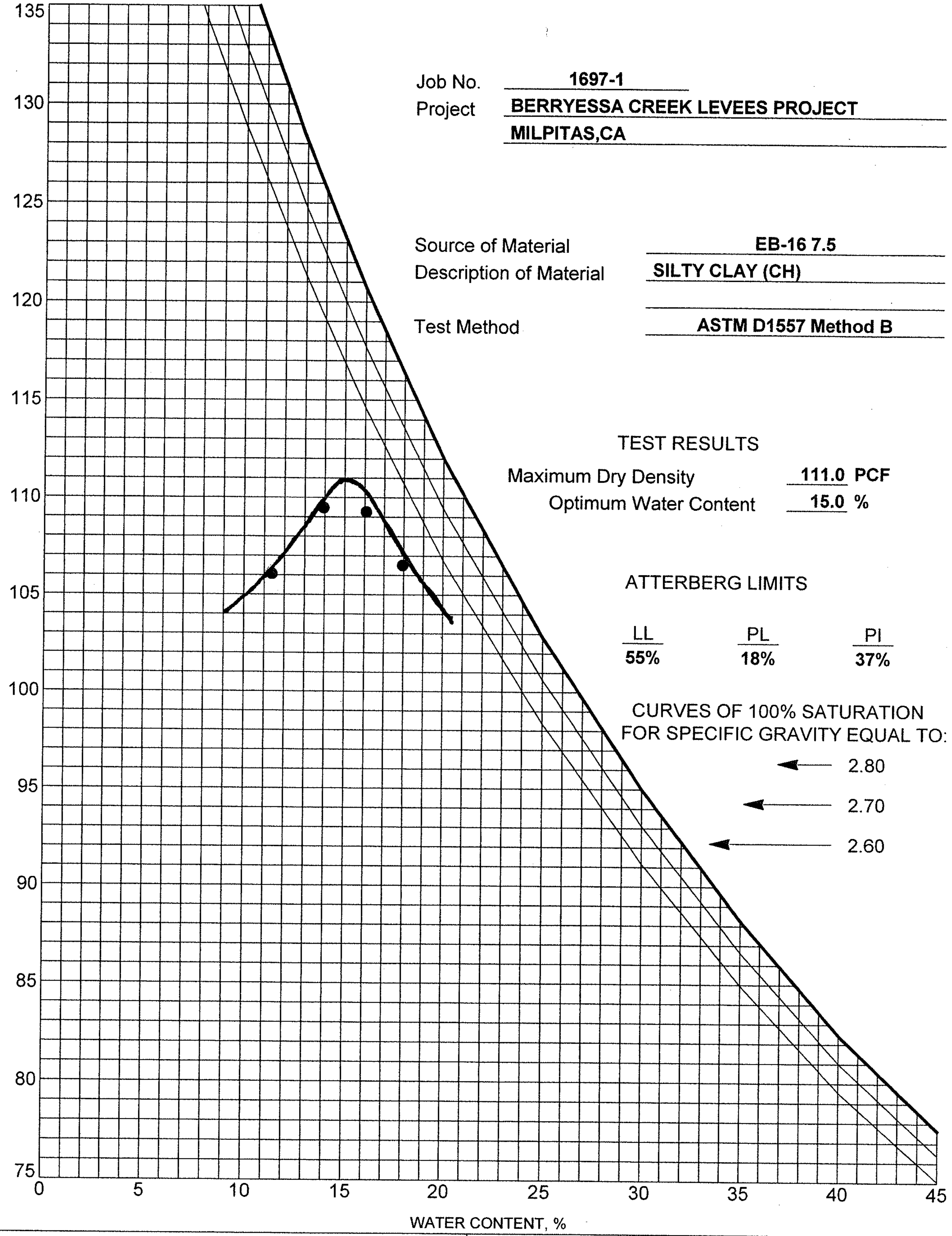
ATTERBERG LIMITS

LL	PL	PI
38%	**15%**	**23%**

MOISTURE-DENSITY RELATIONSHIP

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

Project: BERRYESSA CREEK LEVEES PROJECT
Location: MILPITAS,CA
Project No.: 1697-1



Job No. 1697-1

Project BERRYESSA CREEK LEVEES PROJECT
MILPITAS,CA

Source of Material EB-16 7.5

Description of Material SILTY CLAY (CH)

Test Method ASTM D1557 Method B

TEST RESULTS

Maximum Dry Density 111.0 PCF

Optimum Water Content 15.0 %

ATTERBERG LIMITS

LL	PL	PI
55%	18%	37%

CURVES OF 100% SATURATION FOR SPECIFIC GRAVITY EQUAL TO:

2.80

2.70

2.60

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

MOISTURE-DENSITY RELATIONSHIP

Project: BERRYESSA CREEK LEVEES PROJECT

Location: MILPITAS,CA

Project No.: 1697-1

LA CORP.GDT 8/22/01 MV* FLL

SGI Testing Services, LLC

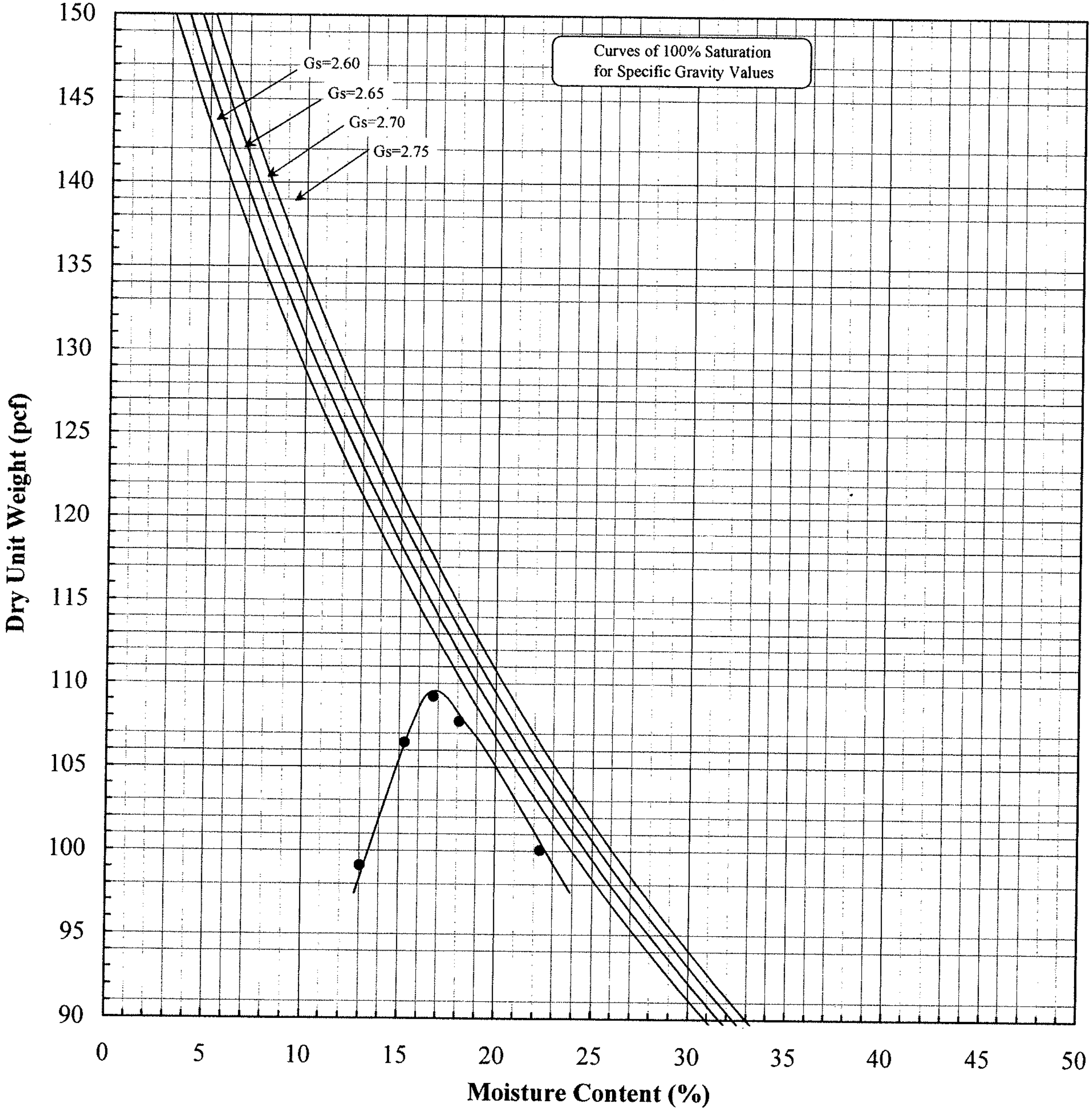
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 3A/B (EB-75)
Lab Sample No:	AL8519

ASTM D1557

COMPACTION MOISTURE-DENSITY RELATIONSHIP

Modified - Method A



Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 3A/B (EB-75)	AL8519	109.5	17.0	

Note(s):

Boring EB-7s
at 4½ - 9 ft.

al8519.soil.proctor.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI Testing Services, LLC

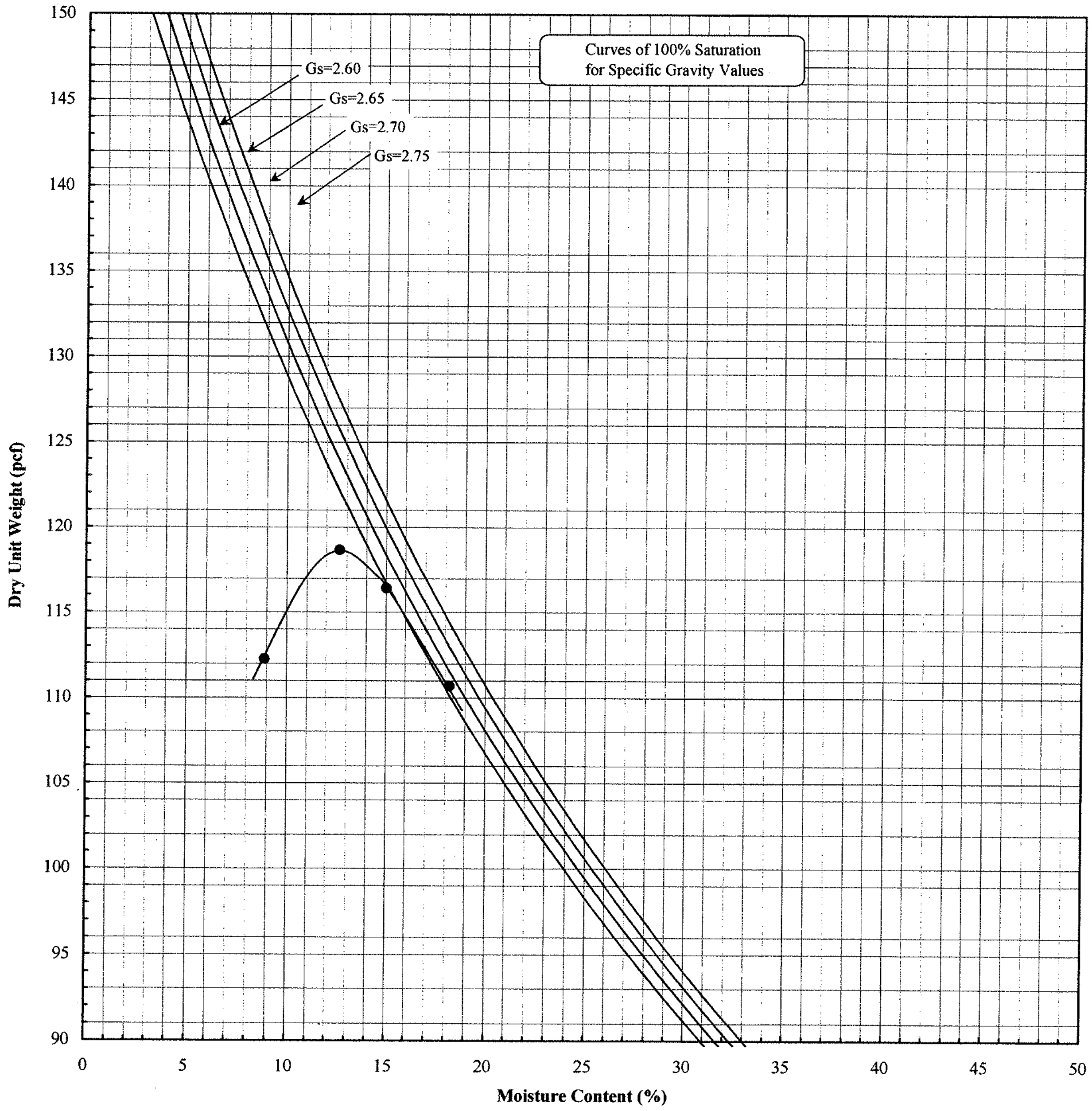
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name: Berryessa Creek Levee
Project No: SGI1021
Client Sample ID: Sample 1A/B (EB-145)
Lab Sample No: AL8517

ASTM D1557

COMPACTION MOISTURE-DENSITY RELATIONSHIP

Modified - Method A



Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 1A/B (EB-145)	AL8517	118.5	12.5	

Note(s):

Boring EB-14s
at 2 - 9 ft.

al8517.soil.proctor.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

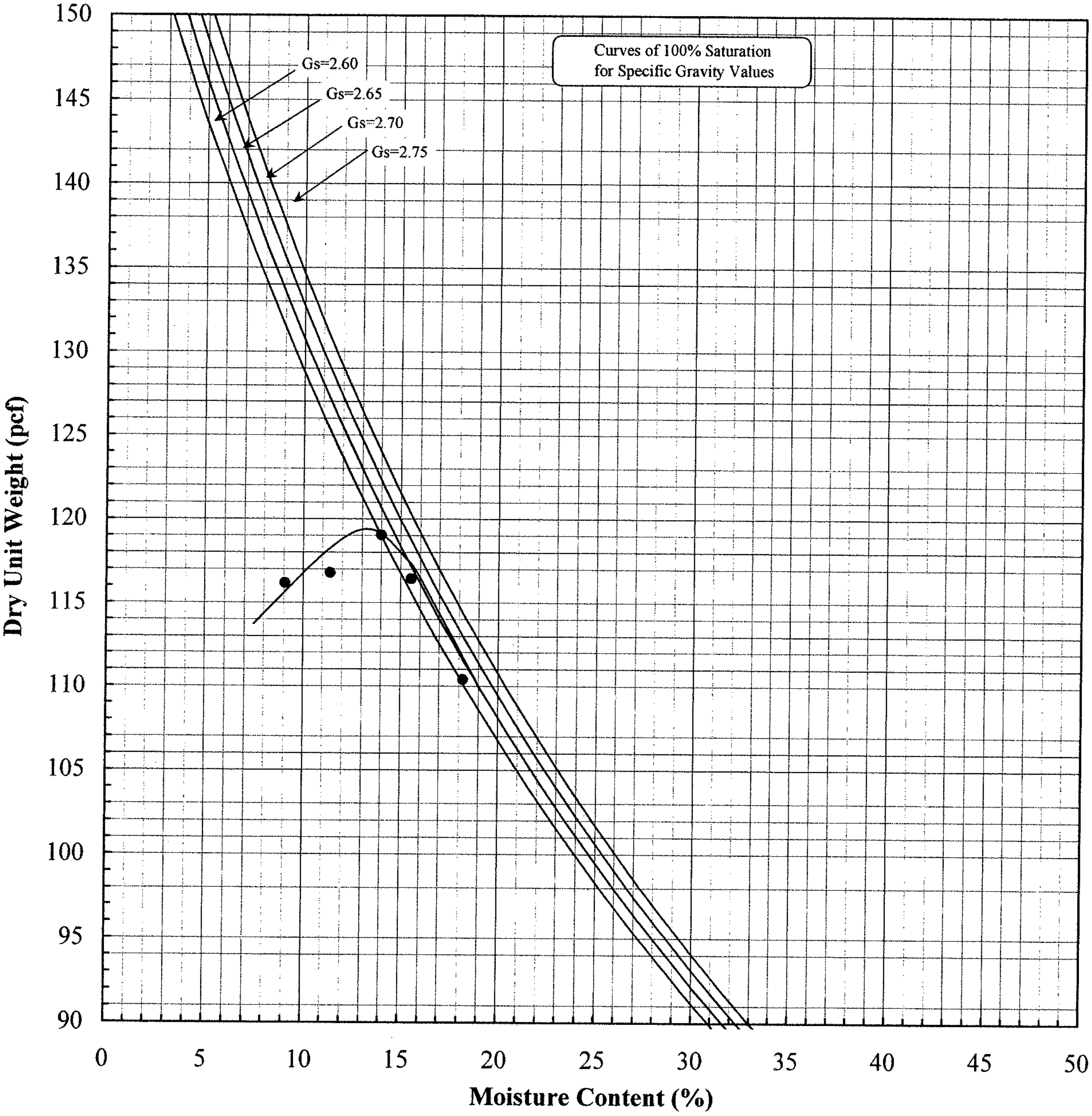
SGI Testing Services, LLC
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 2A/B (EB-155)
Lab Sample No:	AL8518

ASTM D1557

COMPACTION MOISTURE-DENSITY RELATIONSHIP

Modified - Method A



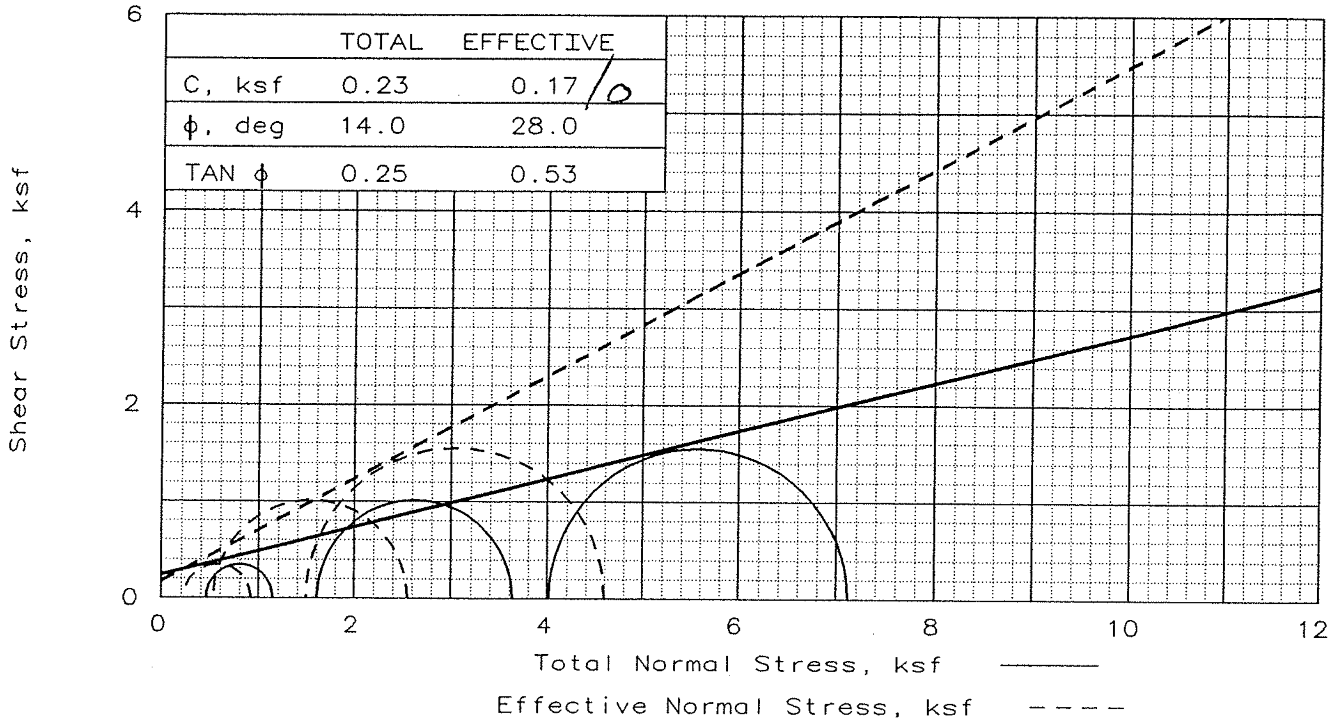
Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 2A/B (EB-155)	AL8518	119.5	13.0	

Note(s):

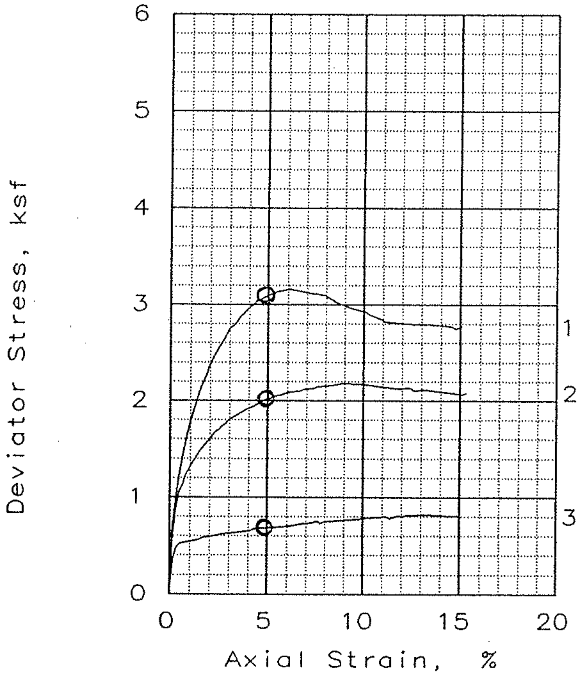
Boring EB-15s
at 1½ - 6½ ft.

al8518.soil.proctor.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO



	TOTAL	EFFECTIVE
C, ksf	0.23	0.17 /0
ϕ, deg	14.0	28.0
TAN ϕ	0.25	0.53



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	27.4	28.8	30.8
	DRY DENSITY, pcf	92.5	93.5	90.9
	SATURATION, %	86.3	90.9	93.4
	VOID RATIO	0.890	0.903	0.924
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	26.3	28.5	34.0
	DRY DENSITY, pcf	100.6	98.1	89.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.737	0.813	0.952
	DIAMETER, in	2.80	2.83	2.89
	HEIGHT, in	5.79	5.90	6.01
Strain rate, %/min		0.03	0.03	0.03
EFF CELL PRESSURE, ksf		4.02	1.61	0.46
DEVIATOR STRESS, ksf		3.10	2.02	0.69
EXCESS PORE PR., ksf		2.52	1.08	0.23
STRAIN, %		5.1	5.2	5.3
ULT. STRESS, ksf				
EXCESS PORE PR., ksf				
STRAIN, %				
$\bar{\sigma}_1$ FAILURE, ksf		4.60	2.56	0.92
$\bar{\sigma}_3$ FAILURE, ksf		1.50	0.53	0.23

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Lt.Brown CLAY tr. sand

ASSUMED SPECIFIC GRAVITY= 2.8
REMARKS: Strengths picked at 5% strain.

Fig. No.: ______

CLIENT: Lowney

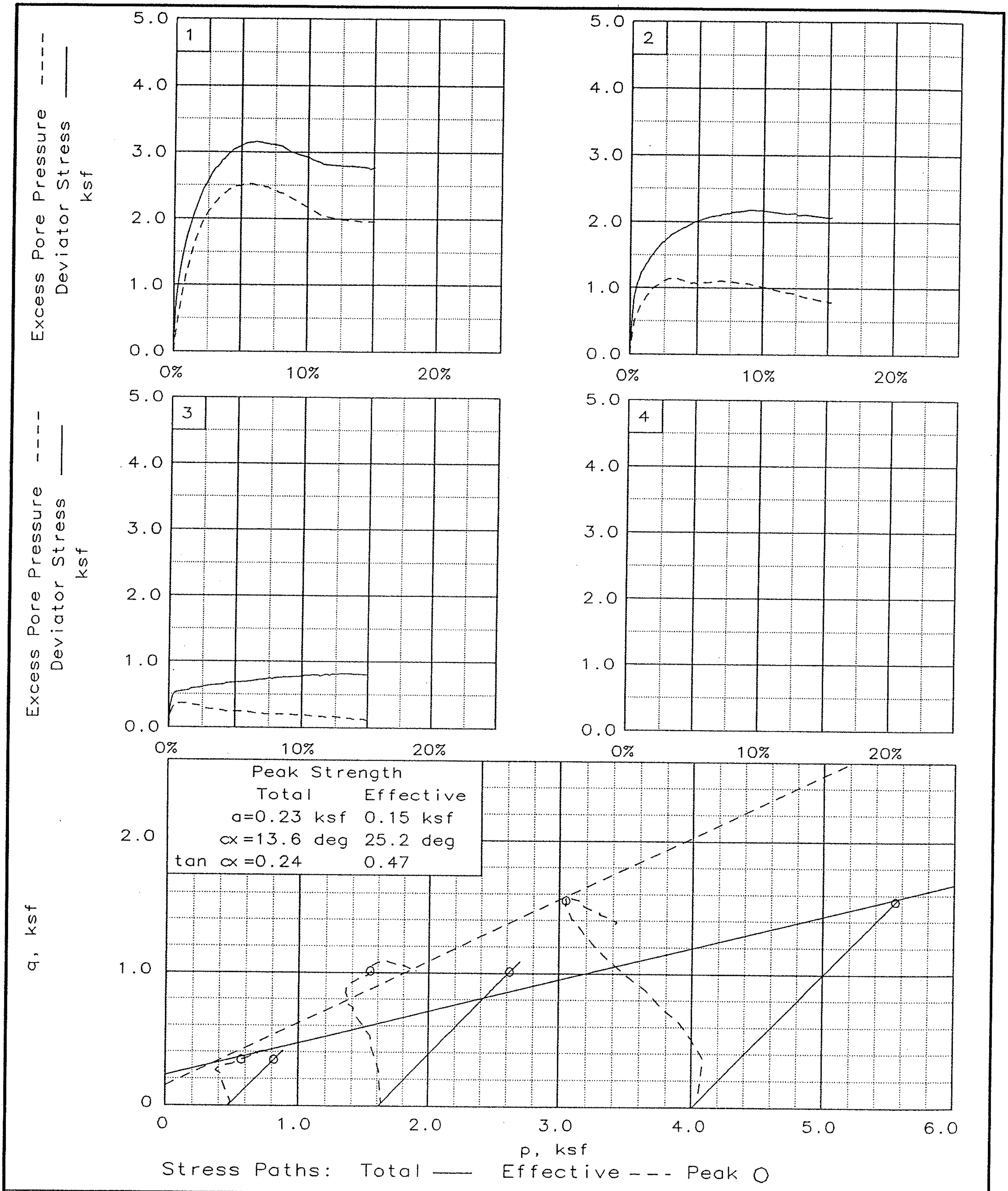
PROJECT: 1697-1

SAMPLE LOCATION: EB1,6A @ 13-16'

PROJ. NO.: 028-962 DATE: 5/18/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



Peak Strength

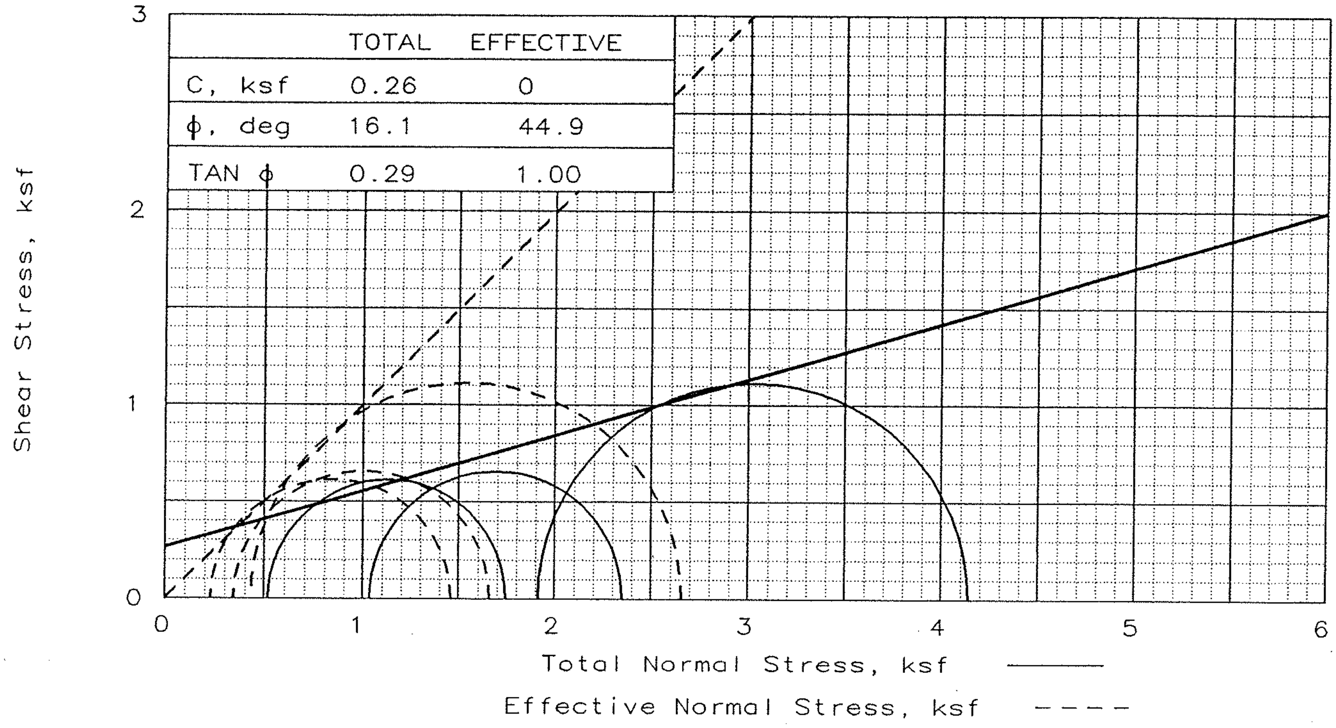
	Total	Effective
a=	0.23 ksf	0.15 ksf
α =	13.6 deg	25.2 deg
tan α =	0.24	0.47

Client: Lowney

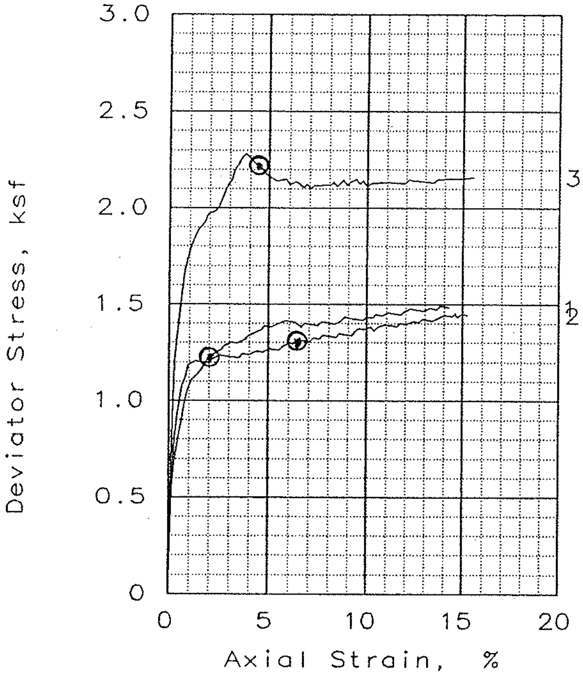
Project: 1697-1

Location: EB1,6A @ 13-16'

File: 028-962 Project No.: 028-962 Fig. No.: _____



	TOTAL	EFFECTIVE
C, ksf	0.26	0
ϕ, deg	16.1	44.9
TAN ϕ	0.29	1.00



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	29.5	30.7	28.8
	DRY DENSITY, pcf	87.0	84.9	89.2
	SATURATION, %	85.1	84.0	87.5
	VOID RATIO	0.937	0.986	0.889
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	32.6	33.4	30.0
	DRY DENSITY, pcf	89.7	88.6	93.2
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.880	0.902	0.809
	DIAMETER, in	2.84	2.82	2.83
	HEIGHT, in	5.96	5.92	5.87
	Strain rate, %/min	0.03	0.03	0.03
	EFF CELL PRESSURE, ksf	0.52	1.04	1.92
	DEVIATOR STRESS, ksf	1.23	1.31	2.23
	EXCESS PORE PR., ksf	0.29	0.69	1.48
	STRAIN, %	2.0	6.4	4.3
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	1.46	1.66	2.66
	$\bar{\sigma}_3$ FAILURE, ksf	0.23	0.35	0.43

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Black CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

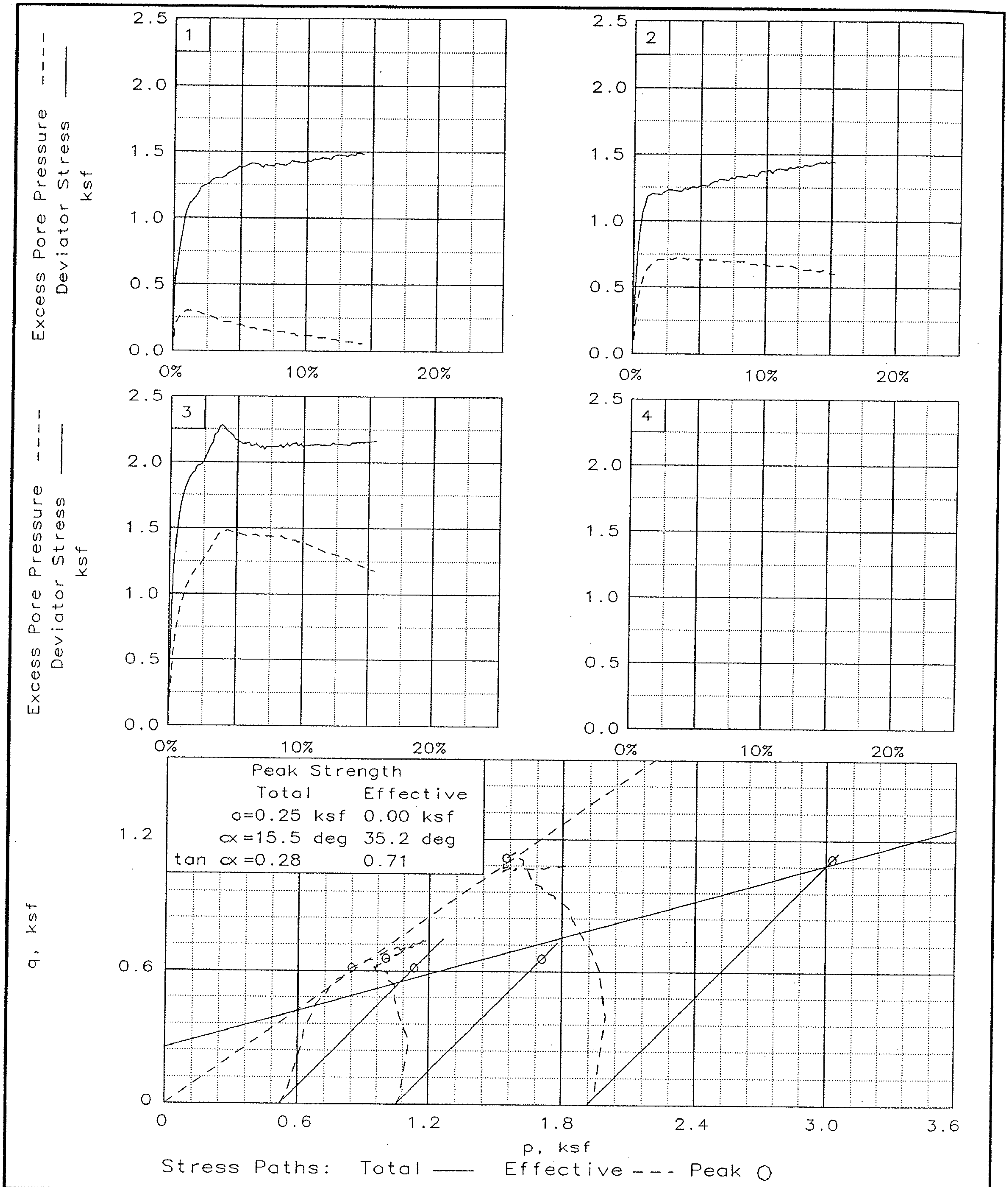
PROJECT: 1697-1

SAMPLE LOCATION: EB3, 2A @ 3.5-6.0'

PROJ. NO.: 028-957 DATE: 5/11/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY

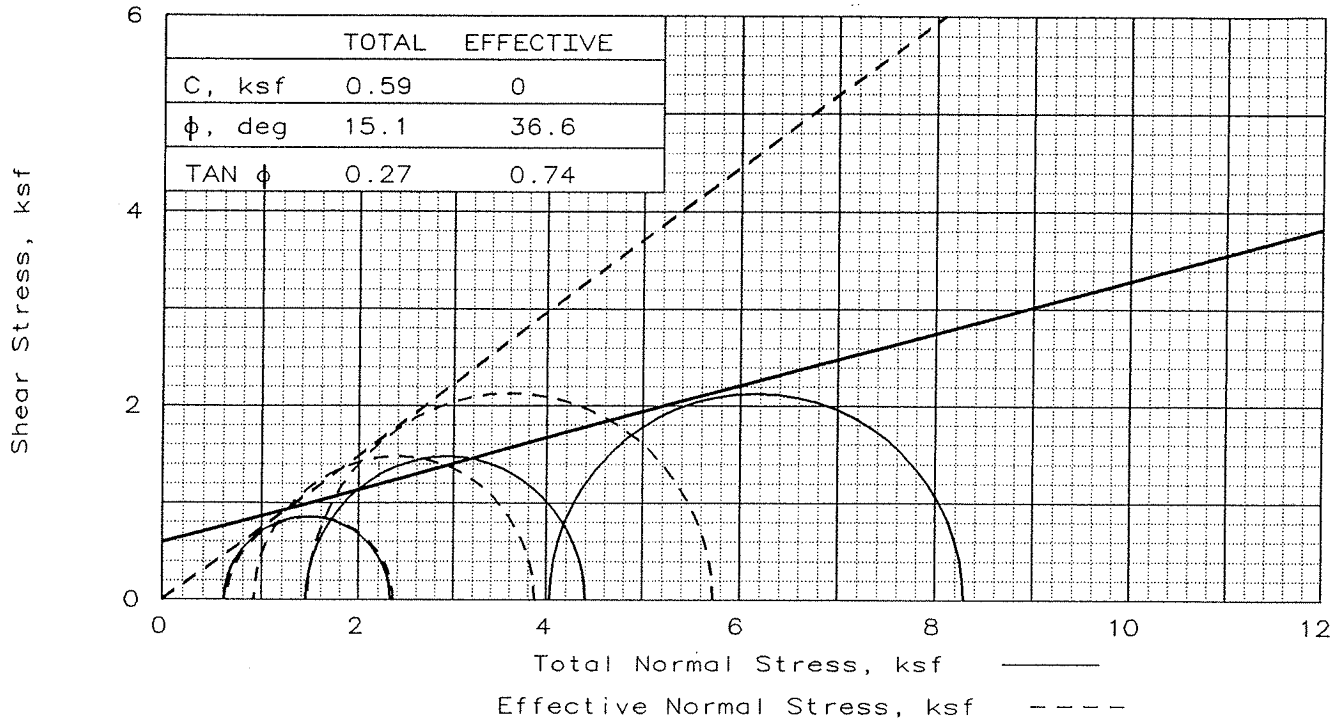


Client: Lowney

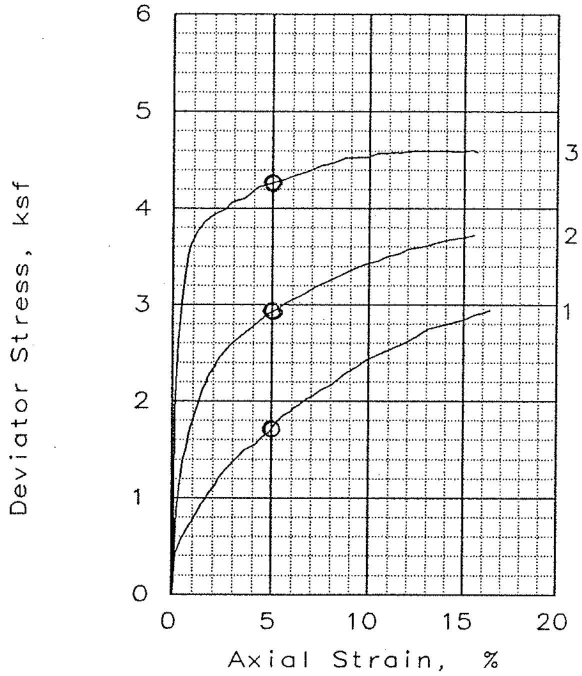
Project: 1697-1

Location: EB3, 2A @ 3.5-6.0'

File: 028-957 Project No.: 028-957 Fig. No.: ____



	TOTAL	EFFECTIVE
C, ksf	0.59	0
ϕ, deg	15.1	36.6
TAN ϕ	0.27	0.74



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	22.9	22.4	21.2
	DRY DENSITY, pcf	102.7	101.6	104.6
	SATURATION, %	96.2	91.8	93.4
	VOID RATIO	0.642	0.659	0.612
	DIAMETER, in	2.86	2.86	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	22.3	21.9	17.0
	DRY DENSITY, pcf	105.2	105.9	115.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.602	0.592	0.460
	DIAMETER, in	2.83	2.82	2.76
	HEIGHT, in	5.96	5.93	5.88
	Strain rate, %/min	0.03	0.03	0.03
	EFF CELL PRESSURE, ksf	0.62	1.45	4.03
	DEVIATOR STRESS, ksf	1.71	2.95	4.26
	EXCESS PORE PR., ksf	-0.03	0.53	2.56
	STRAIN, %	5.0	5.2	4.9
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	2.36	3.87	5.73
	$\bar{\sigma}_3$ FAILURE, ksf	0.65	0.92	1.47

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Lt.brown Clayey
SAND/Sandy CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at 5%
strain.

Fig. No.: ______

CLIENT: Lowney

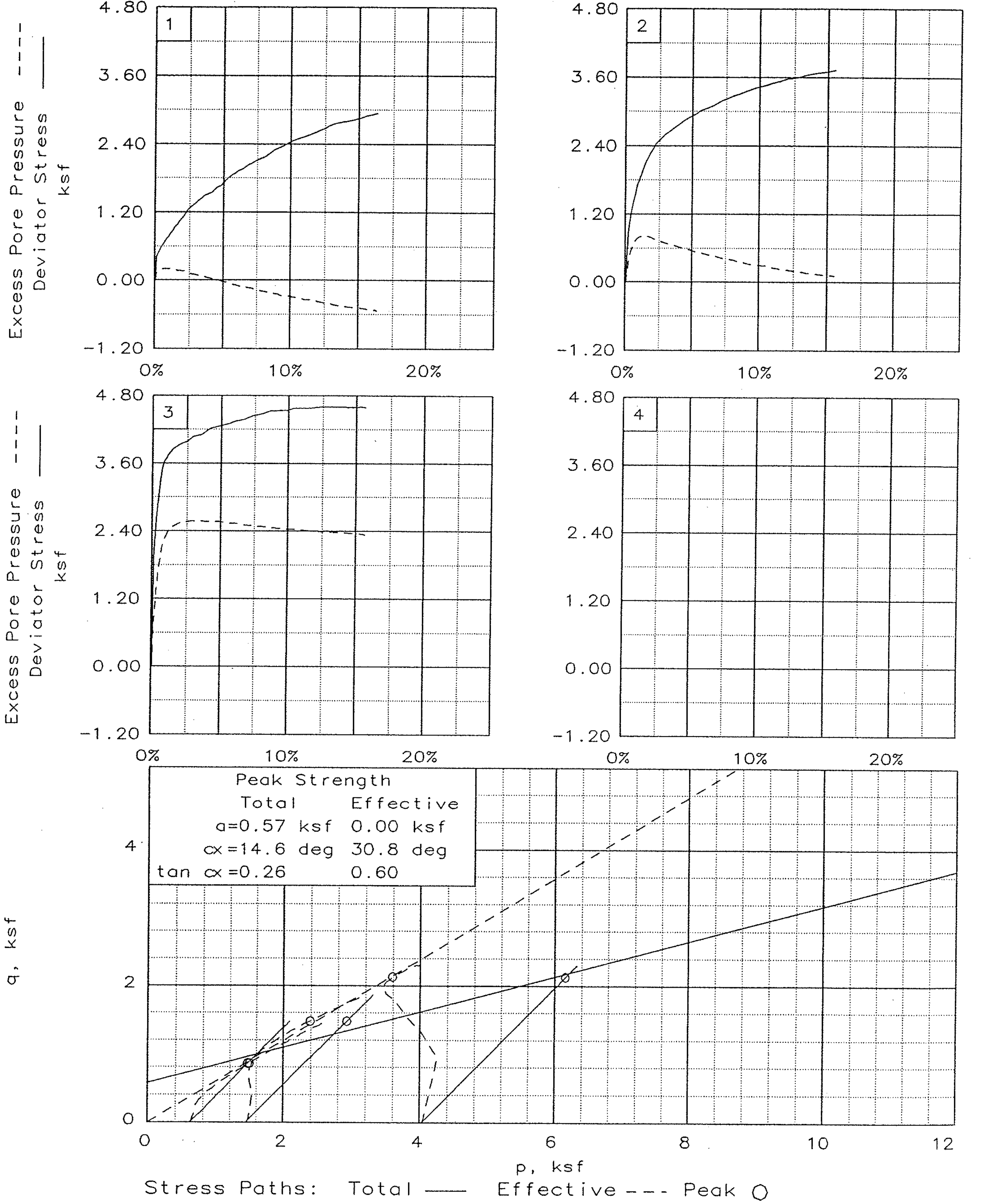
PROJECT: 1697-1

SAMPLE LOCATION: EB3,6A @ 15-18'

PROJ. NO.: 028-962a DATE: 5/21/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



Peak Strength

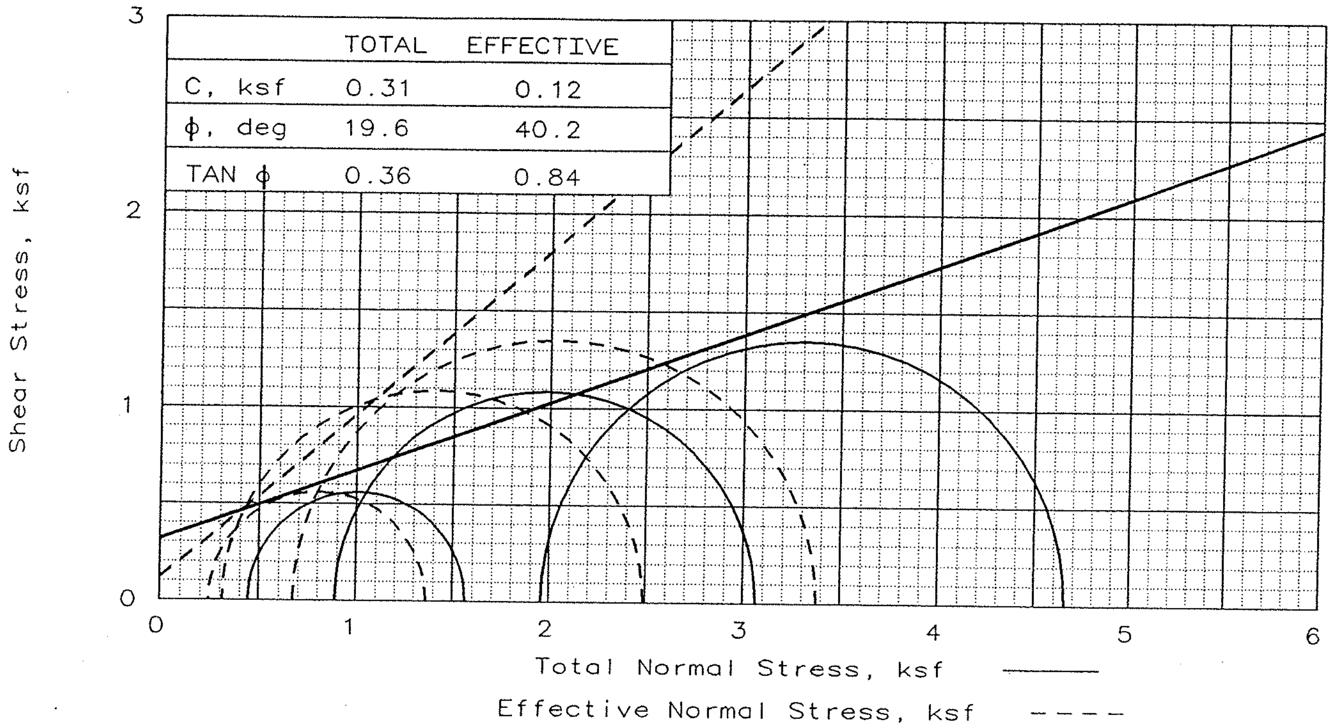
	Total	Effective
a=	0.57 ksf	0.00 ksf
α=	14.6 deg	30.8 deg
tan α=	0.26	0.60

Client: Lowney

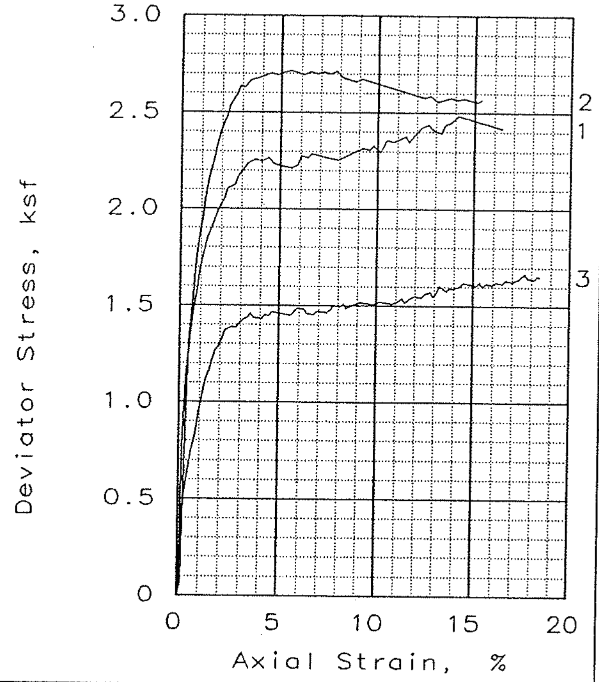
Project: 1697-1

Location: EB3,6A @ 15-18'

File: 028-962A Project No.: 028-962a Fig. No.: _____



	TOTAL	EFFECTIVE
C, ksf	0.31	0.12
ϕ, deg	19.6	40.2
TAN ϕ	0.36	0.84



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	11.6	29.7	27.4
	DRY DENSITY, pcf	104.0	89.9	93.5
	SATURATION, %	50.6	91.7	92.3
	VOID RATIO	0.621	0.874	0.802
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	12.9	30.0	29.5
	DRY DENSITY, pcf	124.9	93.1	93.9
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.349	0.810	0.796
	DIAMETER, in	2.64	2.85	2.87
	HEIGHT, in	5.91	5.89	5.96
	Strain rate, %/min	0.03	0.03	0.03
	EFF CELL PRESSURE, ksf	0.89	1.96	0.45
	DEVIATOR STRESS, ksf	2.17	2.70	1.12
	EXCESS PORE PR., ksf	0.58	1.28	0.20
	STRAIN, %	2.8	4.4	1.3
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	2.49	3.38	1.37
	$\bar{\sigma}_3$ FAILURE, ksf	0.32	0.68	0.24

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Dark Brown CLAY
trace Sand

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at
the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

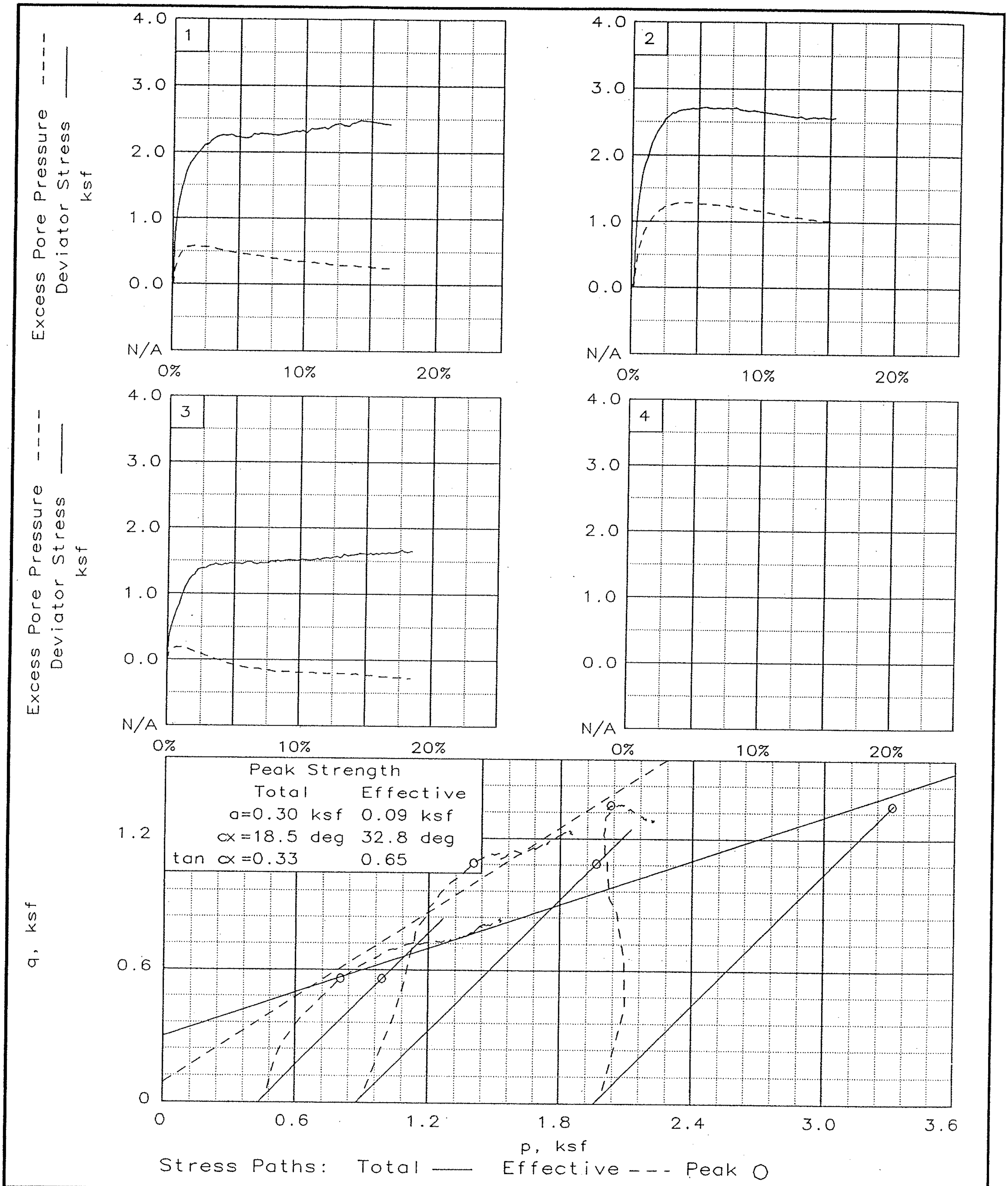
PROJECT: 1697-1

SAMPLE LOCATION: EB7,2A @ 3.0-6.0'

PROJ. NO.: 028-945a DATE: 5/4/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



Peak Strength

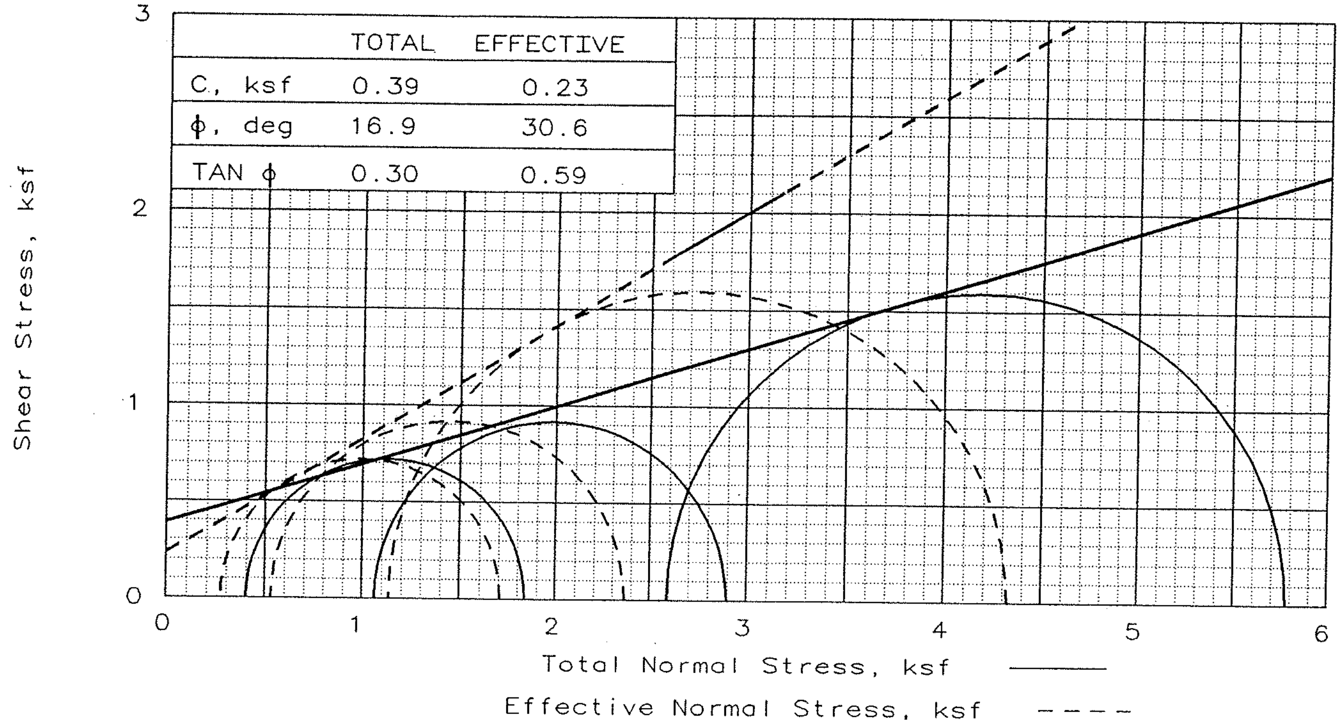
	Total	Effective
a=	0.30 ksf	0.09 ksf
α=	18.5 deg	32.8 deg
tan α=	0.33	0.65

Client: Lowney

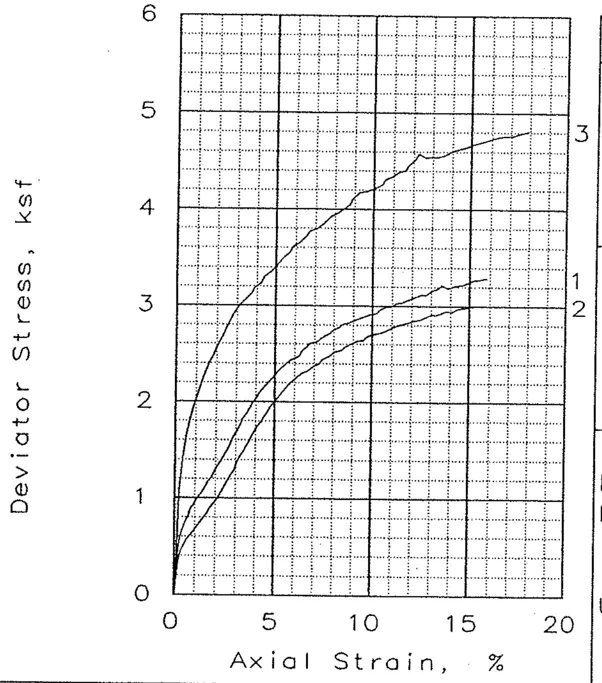
Project: 1697-1

Location: EB7,2A @ 3.0-6.0'

File: 028-945A   Project No.: 028-945a   Fig. No.: ____



	TOTAL	EFFECTIVE
C, ksf	0.39	0.23
ϕ, deg	16.9	30.6
TAN ϕ	0.30	0.59



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	18.6	20.4	18.4
	DRY DENSITY, pcf	108.6	106.6	111.8
	SATURATION, %	90.7	94.6	98.0
	VOID RATIO	0.553	0.582	0.508
	DIAMETER, in	2.87	2.87	2.86
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	18.6	20.7	17.0
	DRY DENSITY, pcf	112.2	108.1	115.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.502	0.559	0.459
	DIAMETER, in	2.85	2.85	2.83
	HEIGHT, in	5.89	5.98	5.91
Strain rate, %/min		0.03	0.03	0.03
EFF CELL PRESSURE, ksf		1.07	0.40	2.59
DEVIATOR STRESS, ksf		1.83	1.44	3.19
EXCESS PORE PR., ksf		0.53	0.13	1.45
STRAIN, %		3.4	3.2	3.9
ULT. STRESS, ksf				
EXCESS PORE PR., ksf				
STRAIN, %				
$\bar{\sigma}_1$ FAILURE, ksf		2.36	1.71	4.32
$\bar{\sigma}_3$ FAILURE, ksf		0.53	0.27	1.14

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Brown Clayey SAND/
Sandy CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at
the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

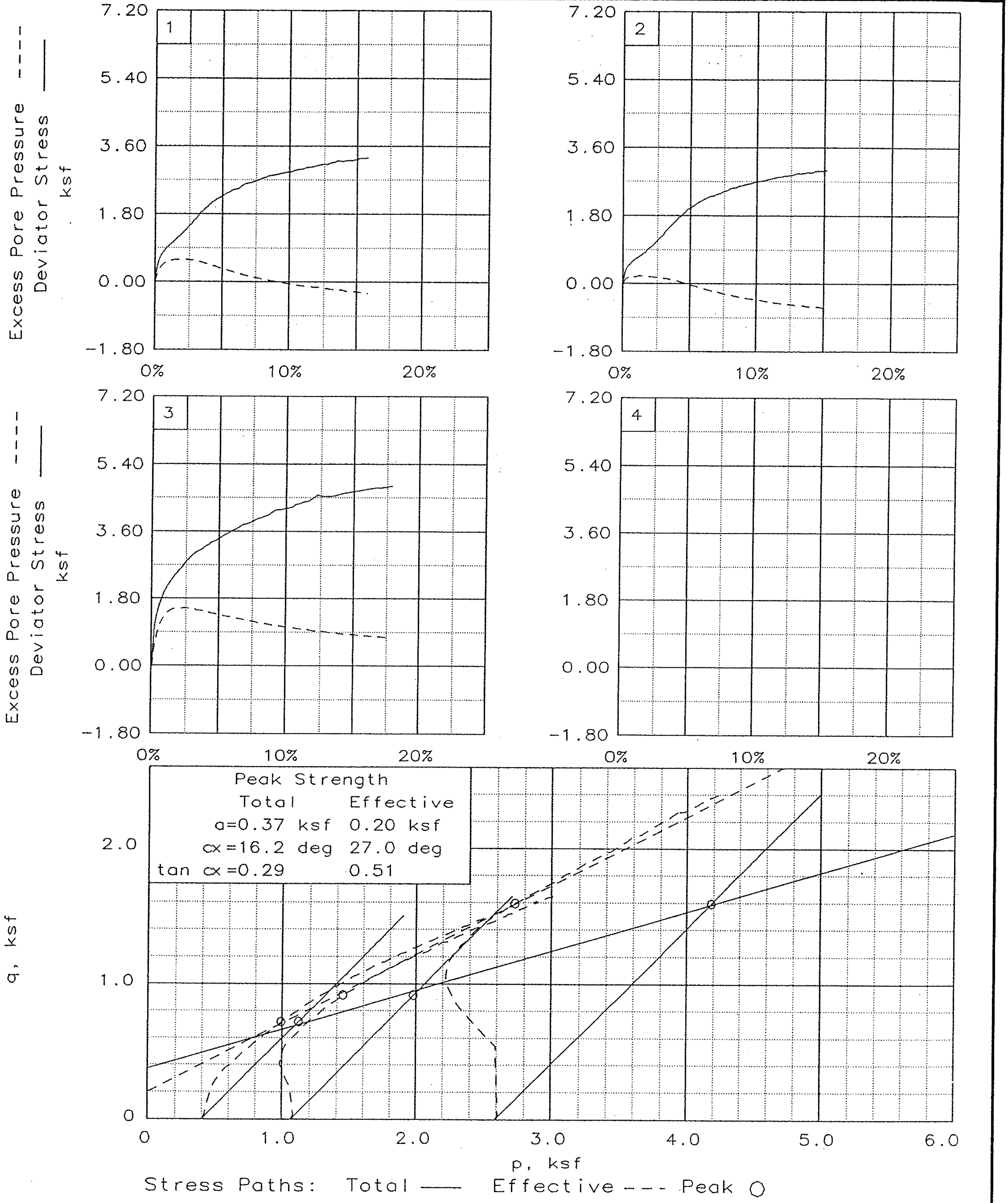
PROJECT: 1697-1

SAMPLE LOCATION: EB7,7A @ 16-19'

PROJ. NO.: 028-945a DATE: 4/30/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



Peak Strength

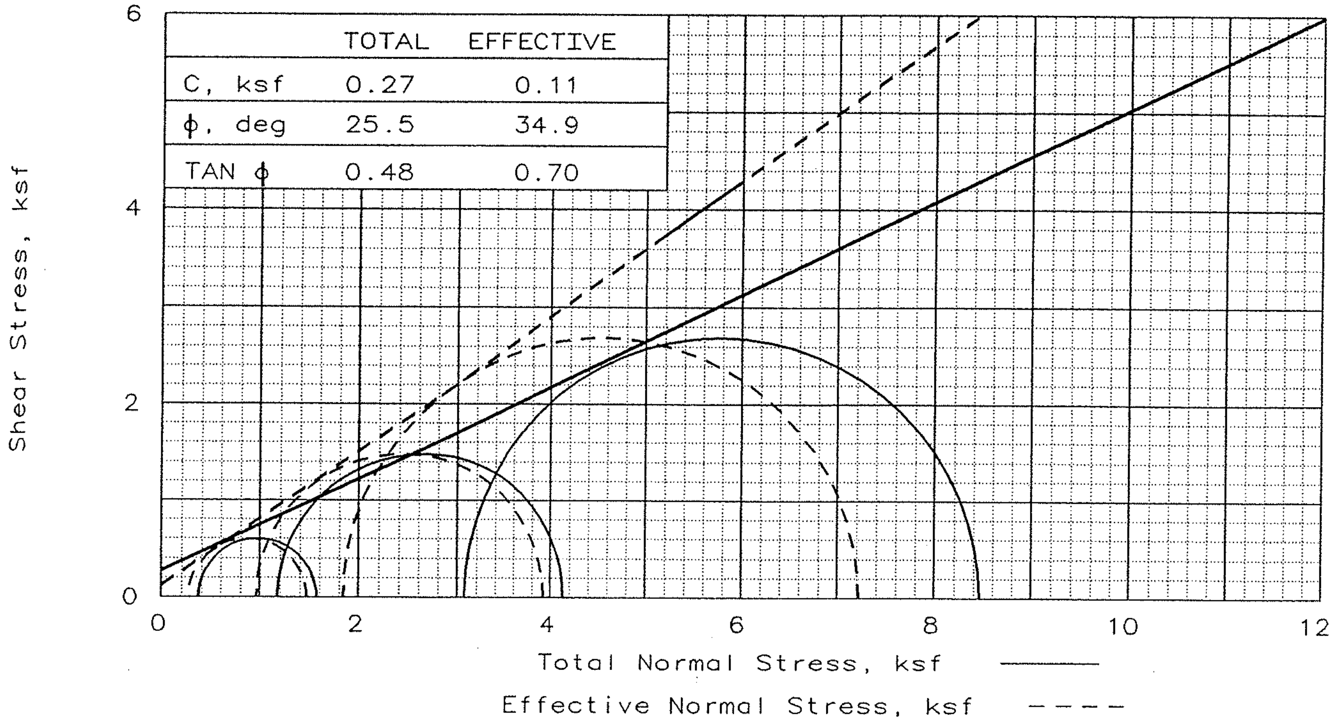
	Total	Effective
a=	0.37 ksf	0.20 ksf
α=	16.2 deg	27.0 deg
tan α=	0.29	0.51

Client: Lowney

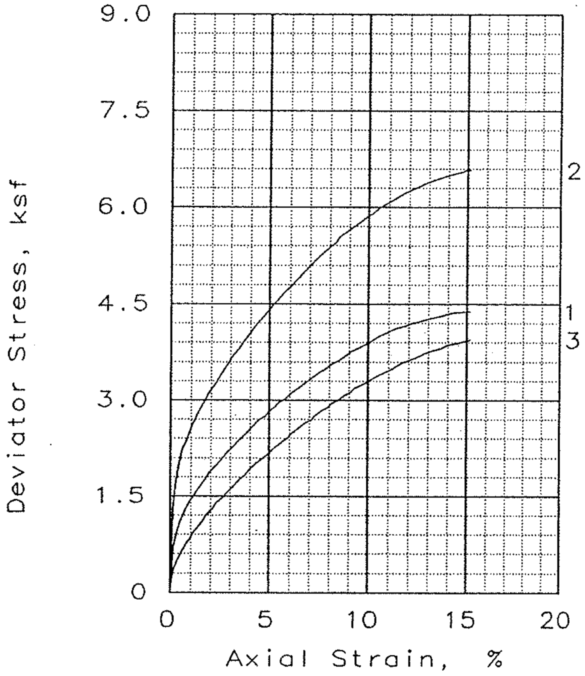
Project: 1697-1

Location: EB7,7A @ 16-19'

File: 028-945 Project No.: 028-945a Fig. No.: ____



	TOTAL	EFFECTIVE
C, ksf	0.27	0.11
ϕ, deg	25.5	34.9
TAN ϕ	0.48	0.70



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	25.4	26.1	24.6
	DRY DENSITY, pcf	97.0	98.1	101.8
	SATURATION, %	93.2	98.3	98.6
	VOID RATIO	0.737	0.718	0.687
	DIAMETER, in	2.87	2.87	2.86
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	23.3	23.3	23.4
	DRY DENSITY, pcf	103.4	103.5	104.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.630	0.629	0.643
	DIAMETER, in	2.78	2.80	2.82
	HEIGHT, in	6.00	5.98	6.00
	Strain rate, %/min	0.03	0.03	0.03
	EFF CELL PRESSURE, ksf	1.18	3.08	0.37
	DEVIATOR STRESS, ksf	2.96	5.38	1.20
	EXCESS PORE PR., ksf	0.22	1.24	0.10
	STRAIN, %	5.5	8.0	1.8
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	3.92	7.22	1.47
	$\bar{\sigma}_3$ FAILURE, ksf	0.96	1.84	0.27

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Brown Silty SAND
close to Sandy SILT

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at
the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

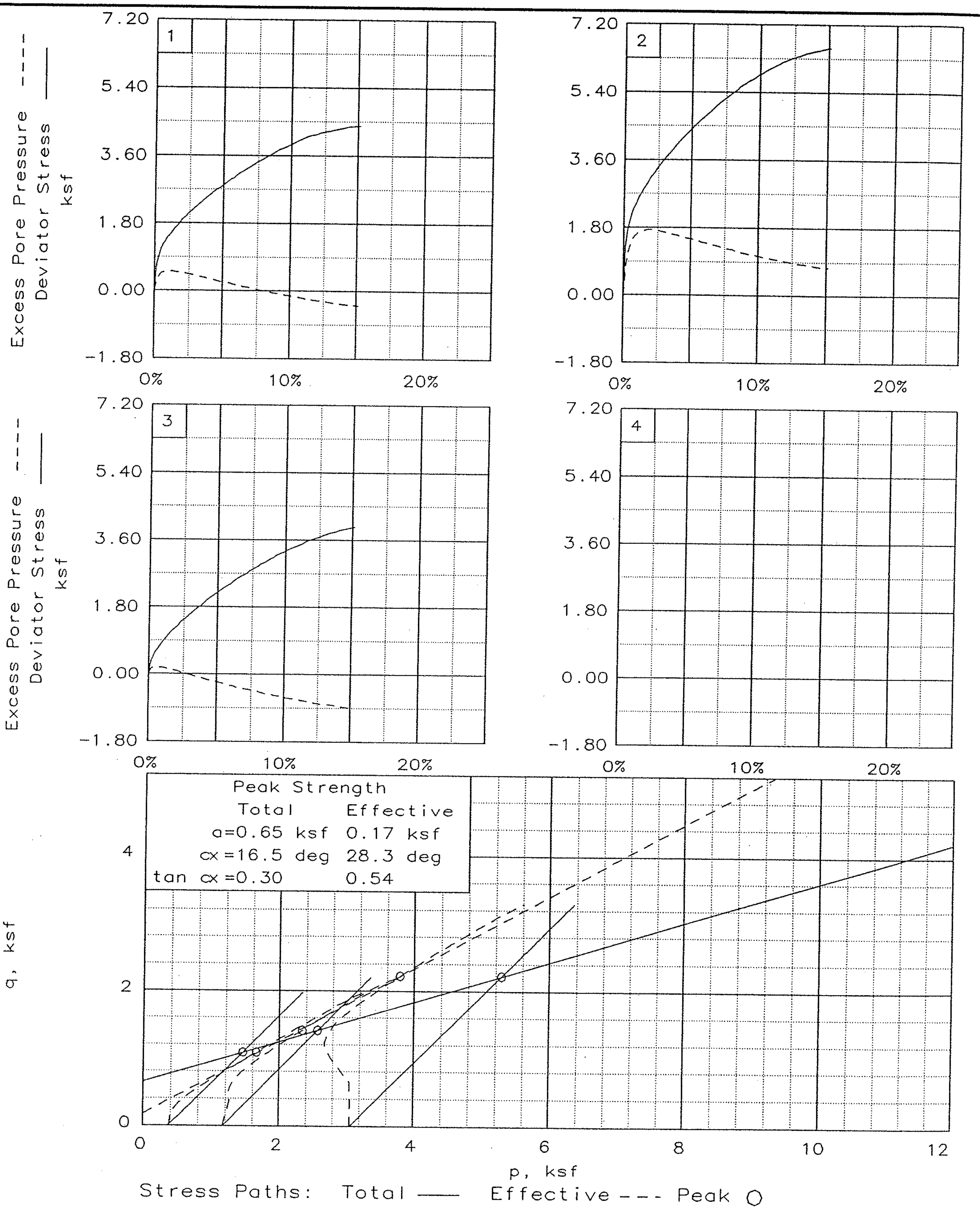
PROJECT: 1697-1

SAMPLE LOCATION: EB9,5A @ 12-15'

PROJ. NO.: 028-962b DATE: 5/23/01

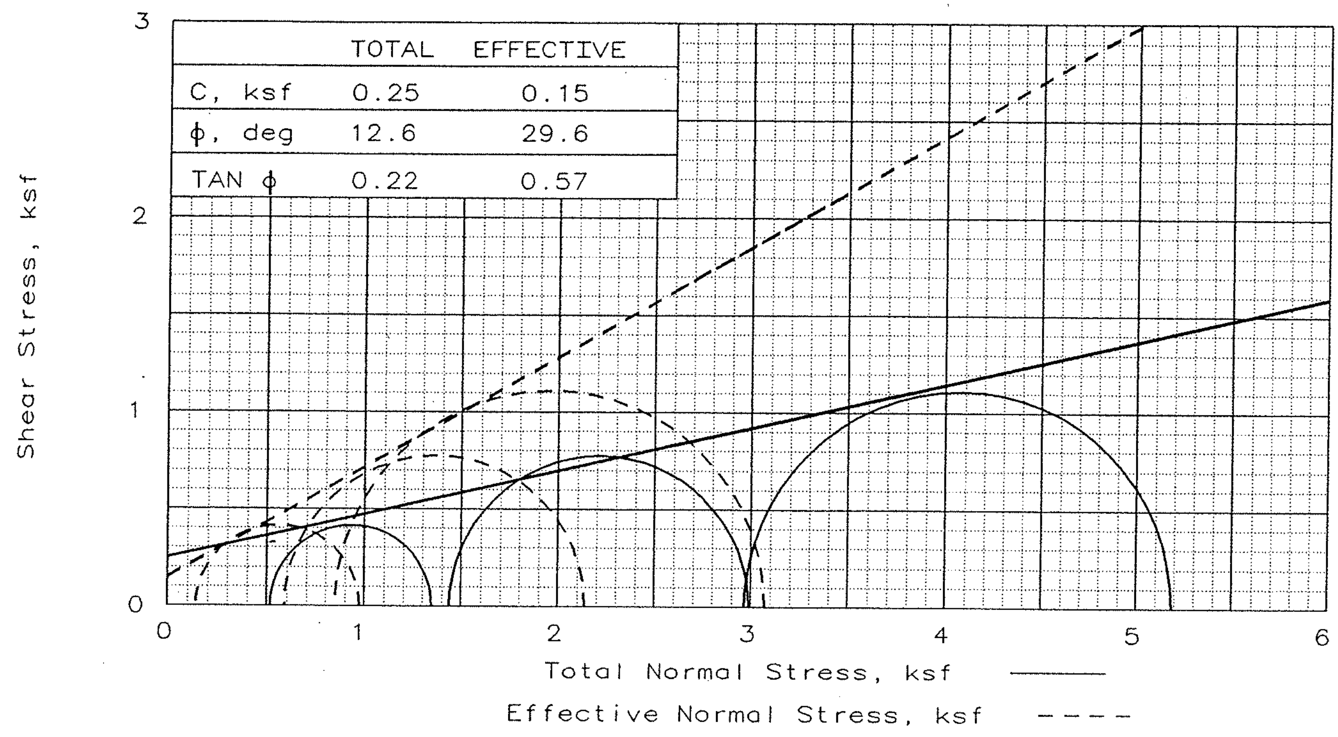
TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY

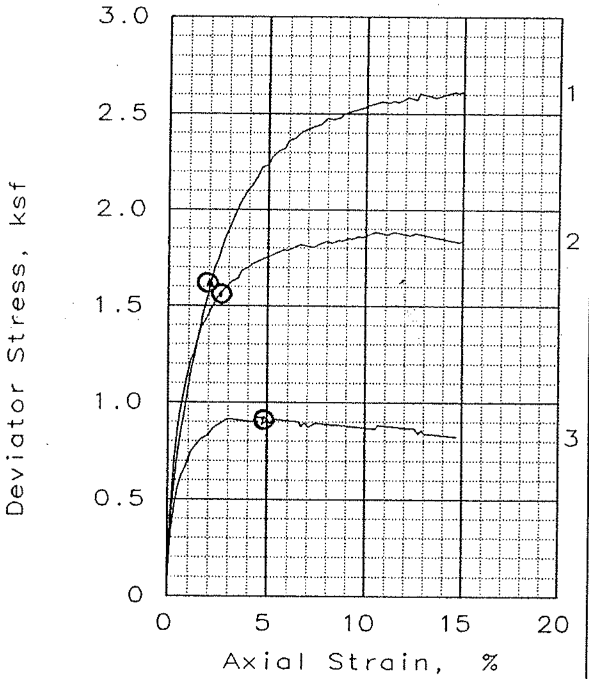


Peak Strength	Total	Effective
a=	0.65 ksf	0.17 ksf
α=	16.5 deg	28.3 deg
tan α=	0.30	0.54

Client: Lowney
Project: 1697-1
Location: EB9,5A @ 12-15'
File: 028-962B Project No.: 028-962b Fig. No.: _____



	TOTAL	EFFECTIVE
C, ksf	0.25	0.15
ϕ, deg	12.6	29.6
TAN ϕ	0.22	0.57



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	18.9	22.3	20.3
	DRY DENSITY, pcf	109.2	103.6	99.7
	SATURATION, %	93.9	96.1	79.5
	VOID RATIO	0.544	0.627	0.690
	DIAMETER, in	2.85	2.85	2.85
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	19.5	23.4	23.5
	DRY DENSITY, pcf	110.3	103.3	103.2
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.528	0.631	0.634
	DIAMETER, in	2.85	2.86	2.79
	HEIGHT, in	5.97	5.98	6.07
Strain rate, %/min		0.02	0.02	0.02
EFF CELL PRESSURE, ksf		2.97	1.44	0.52
DEVIATOR STRESS, ksf		2.22	1.55	0.83
EXCESS PORE PR., ksf		2.12	0.85	0.37
STRAIN, %		4.7	2.5	2.0
ULT. STRESS, ksf				
EXCESS PORE PR., ksf				
STRAIN, %				
$\bar{\sigma}_1$ FAILURE, ksf		3.07	2.14	0.97
$\bar{\sigma}_3$ FAILURE, ksf		0.85	0.59	0.14

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Brown, mottled grey Sandy CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths taken at the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

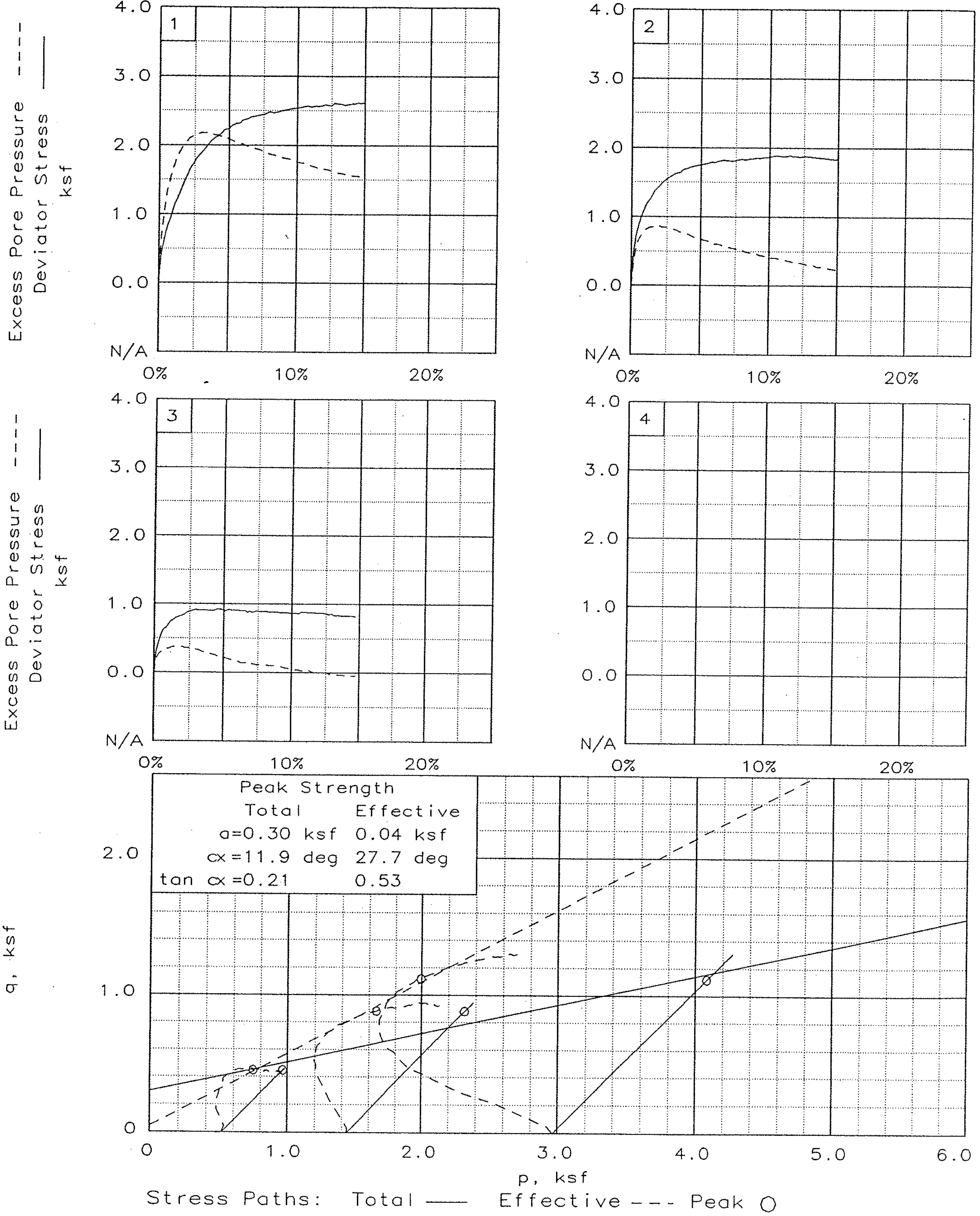
PROJECT: 1697-1

SAMPLE LOCATION: EB15,4A @ 11-14'

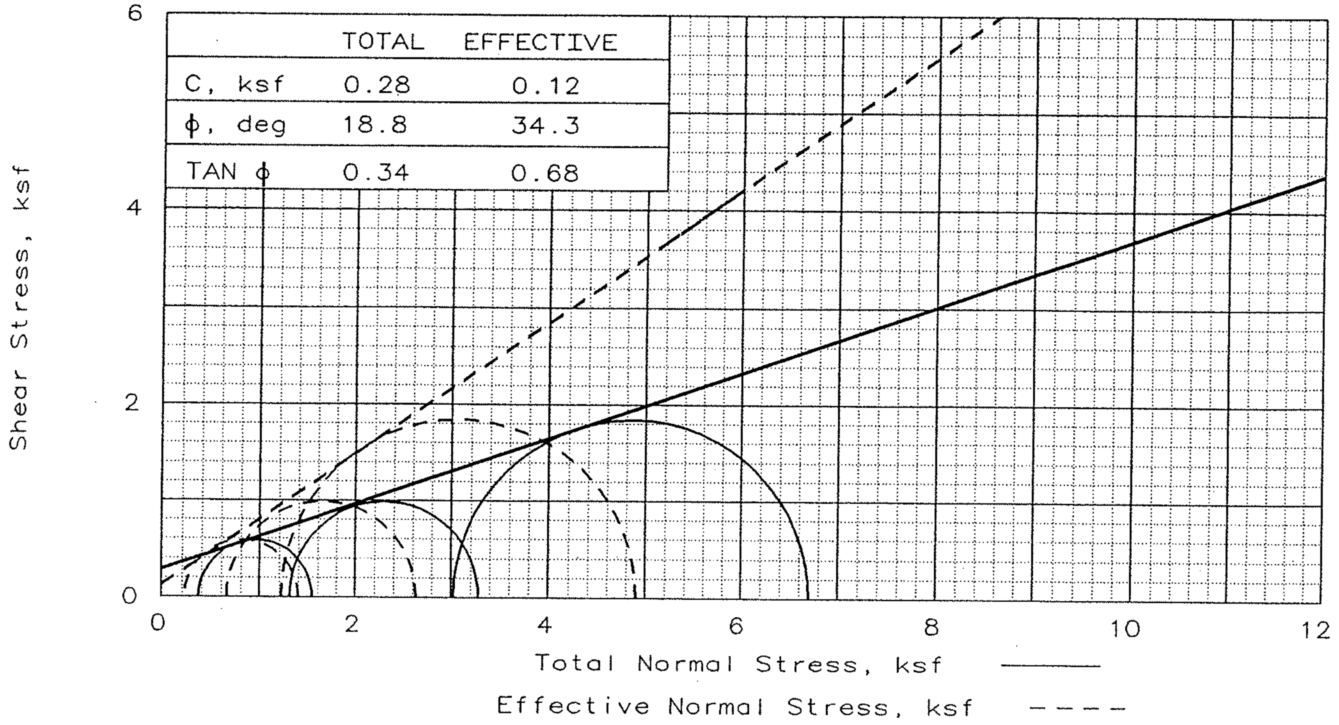
PROJ. NO.: 028-975 DATE: 06/13/01

TRIAXIAL SHEAR TEST REPORT

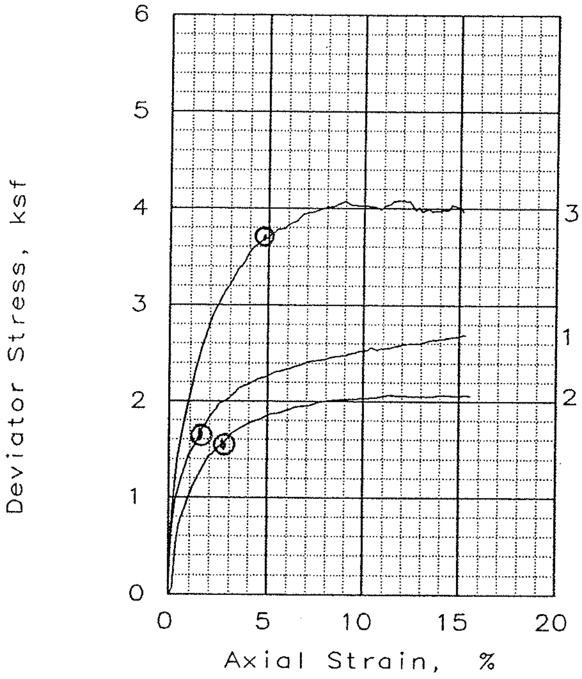
COOPER TESTING LABORATORY



Client: Lowney
Project: 1697-1
Location: EB15,4A @ 11-14'
File: 028-975 Project No.: 028-975 Fig. No.: _____



	TOTAL	EFFECTIVE
C, ksf	0.28	0.12
φ, deg	18.8	34.3
TAN φ	0.34	0.68



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	25.5	23.0	25.0
	DRY DENSITY, pcf	96.3	100.3	98.7
	SATURATION, %	91.8	91.1	95.2
	VOID RATIO	0.750	0.680	0.708
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	26.5	24.6	25.5
	DRY DENSITY, pcf	98.3	101.3	99.9
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.715	0.664	0.687
	DIAMETER, in	2.85	2.86	2.86
	HEIGHT, in	5.95	5.98	5.95
Strain rate, %/min		0.03	0.03	0.03
EFF CELL PRESSURE, ksf		1.31	0.37	3.02
DEVIATOR STRESS, ksf		1.98	1.17	3.68
EXCESS PORE PR., ksf		0.65	0.14	1.80
STRAIN, %		2.5	1.3	4.8
ULT. STRESS, ksf				
EXCESS PORE PR., ksf				
STRAIN, %				
$\bar{\sigma}_1$ FAILURE, ksf		2.64	1.40	4.91
$\bar{\sigma}_3$ FAILURE, ksf		0.66	0.23	1.22

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: Drk.Brn.,mottled
Lt.Brn. CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: Strengths picked at
the peak stress ratio.

Fig. No.: ______

CLIENT: Lowney

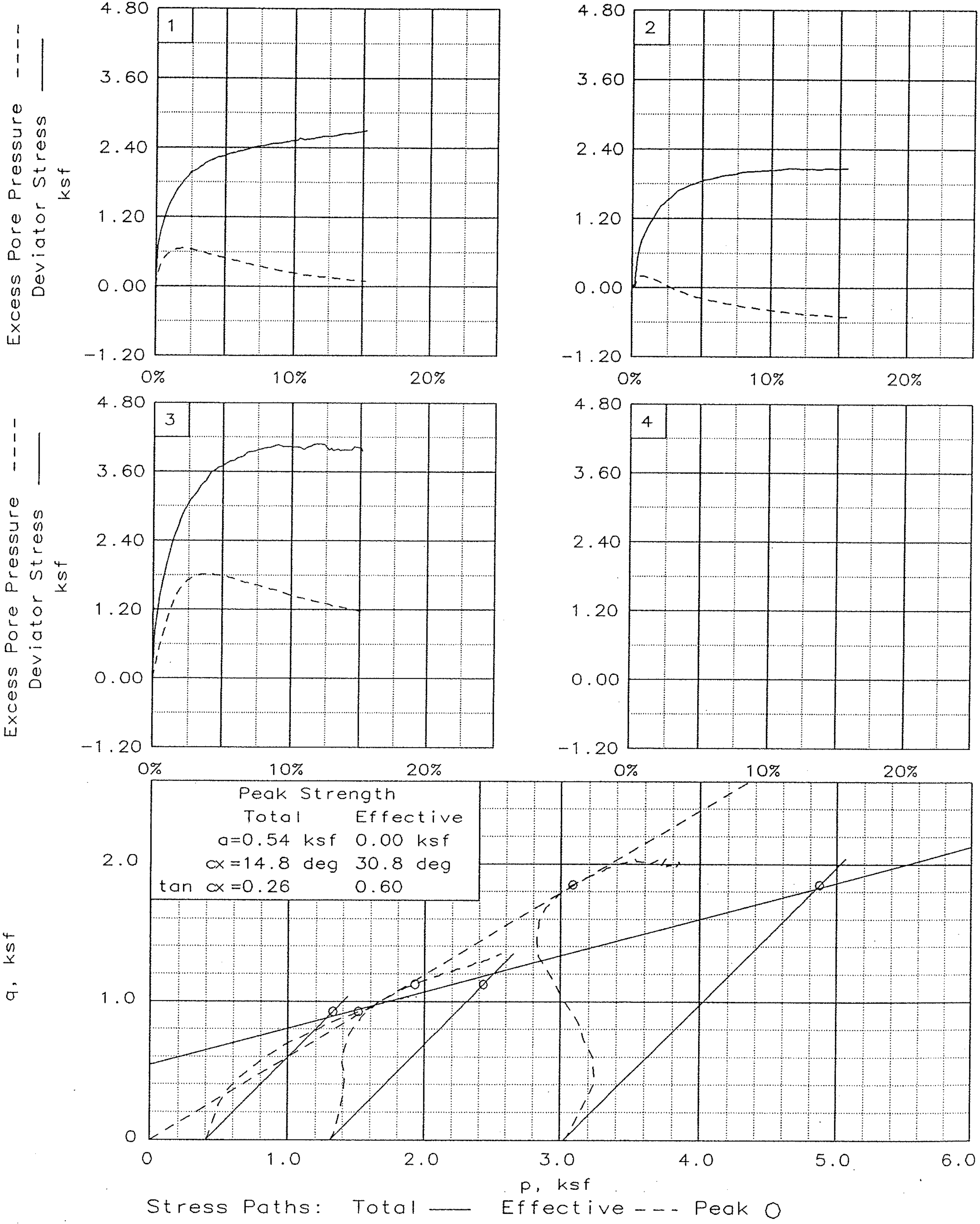
PROJECT: 1697-1

SAMPLE LOCATION: EB16,3A @ 5-8'

PROJ. NO.: 028-957a DATE: 5/16/01

TRIAXIAL SHEAR TEST REPORT

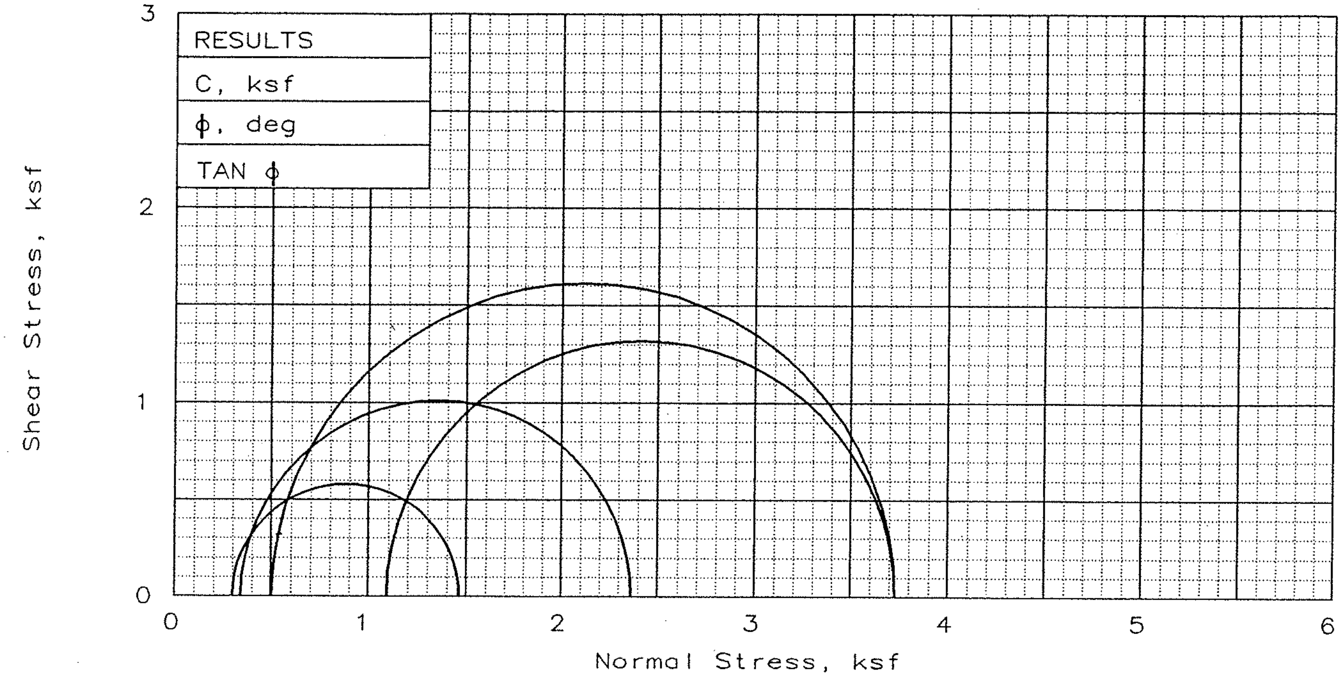
COOPER TESTING LABORATORY

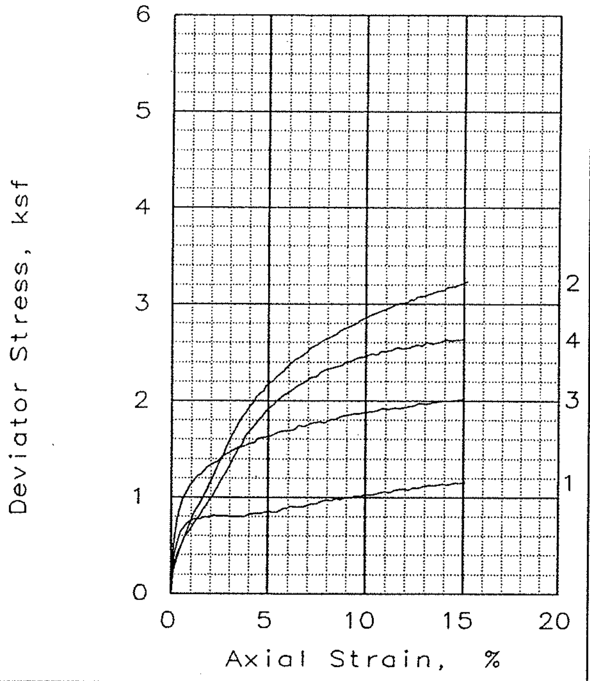


Peak Strength

	Total	Effective
a=	0.54 ksf	0.00 ksf
∝=	14.8 deg	30.8 deg
tan ∝=	0.26	0.60

Client: Lowney
Project: 1697-1
Location: EB16,3A @ 5-8'
File: 028-957A Project No.: 028-957a Fig. No.: _____





SAMPLE NO.:		1	2	3	4
INITIAL	WATER CONTENT, %	19.7	24.6	24.2	21.4
	DRY DENSITY, pcf	99.6	99.1	99.6	106.1
	SATURATION, %	76.8	90.2	89.8	92.6
	VOID RATIO	0.692	0.764	0.755	0.647
	DIAMETER, in	2.86	2.87	2.86	2.87
	HEIGHT, in	6.00	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	25.3	27.3	26.8	22.9
	DRY DENSITY, pcf	99.6	99.1	99.6	106.1
	SATURATION, %	98.9	99.9	99.3	99.1
	VOID RATIO	0.692	0.764	0.755	0.647
	DIAMETER, in	2.86	2.87	2.86	2.87
	HEIGHT, in	6.00	6.00	6.00	6.00
Strain rate, %/min		1.00	1.00	1.00	1.00
BACK PRESSURE, ksf		6.98	5.54	5.54	5.54
CELL PRESSURE, ksf		7.29	6.05	5.89	6.64
DEVIATOR STRESS, ksf		1.16	3.23	2.02	2.64
STRAIN, %		15.1	15.2	15.0	15.0
ULT. STRESS, ksf					
STRAIN, %					
σ_1 FAILURE, ksf		1.46	3.73	2.37	3.73
σ_3 FAILURE, ksf		0.30	0.50	0.35	1.09

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: undisturbed
DESCRIPTION: brown CLAY w/sand

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: The samples were BP-Sat. The cell pressure was increased with the drainage valves closed. TX-UU-Sat.
Fig. No.: ______

CLIENT: Lowney

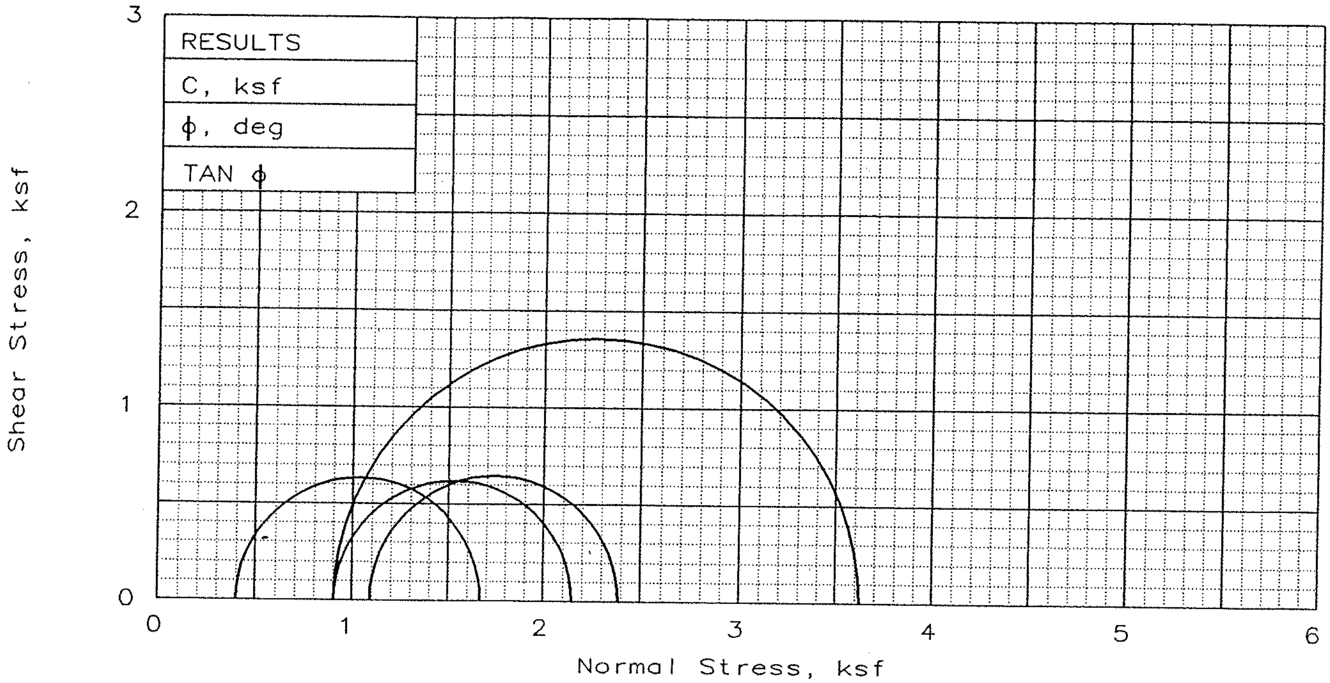
PROJECT: 1697-1

SAMPLE LOCATION: 1)EB1 @ 5' 2)EB4 @ 5' 3)EB11 @ 5.5' 4)EB14 @ 15'

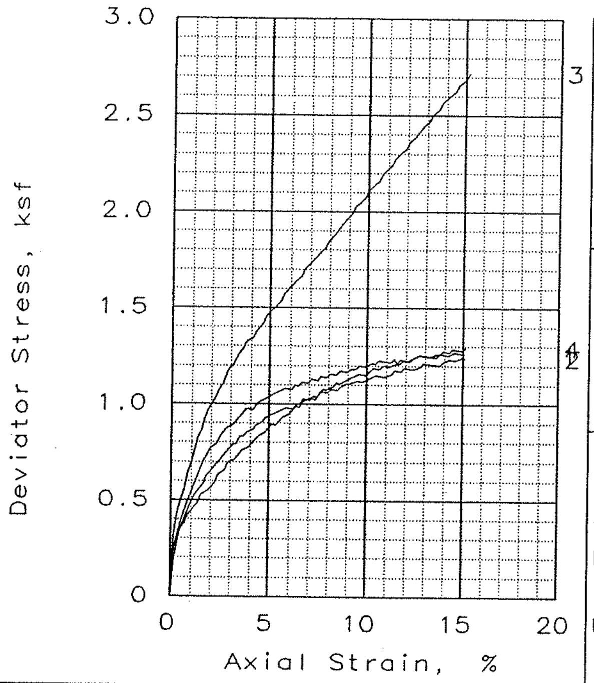
PROJ. NO.: 028-975d DATE: 6/22/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



RESULTS	
C, ksf	
φ, deg	
TAN φ	



SAMPLE NO.:		1	2	3	4
INITIAL	WATER CONTENT, %	26.9	23.3	29.7	31.6
	DRY DENSITY, pcf	91.8	100.2	94.4	86.8
	SATURATION, %	86.9	87.5	99.7	90.7
	VOID RATIO	0.836	0.745	0.818	0.942
	DIAMETER, in	2.86	2.88	2.85	2.86
	HEIGHT, in	6.00	6.00	5.99	6.00
AT TEST	WATER CONTENT, %	30.8	26.6	29.7	33.2
	DRY DENSITY, pcf	91.8	100.2	94.4	86.8
	SATURATION, %	99.4	100.0	99.9	95.3
	VOID RATIO	0.836	0.745	0.818	0.942
	DIAMETER, in	2.86	2.88	2.85	2.86
	HEIGHT, in	6.00	6.00	5.99	6.00
Strain rate, %/min		1.00	1.00	1.00	1.00
BACK PRESSURE, ksf		6.98	5.54	5.54	5.54
CELL PRESSURE, ksf		7.39	6.45	6.45	6.64
DEVIATOR STRESS, ksf		1.27	1.24	2.72	1.29
STRAIN, %		14.6	14.9	15.2	15.1
ULT. STRESS, ksf					
STRAIN, %					
σ_1 FAILURE, ksf		1.67	2.15	3.63	2.39
σ_3 FAILURE, ksf		0.40	0.91	0.91	1.09

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE:
DESCRIPTION: brown CLAY w/sand

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: The samples were BP-Saturated. The confining pressure was added with the valves closed. TX-UU-SAT.

Fig. No.: ______

CLIENT: Lowney

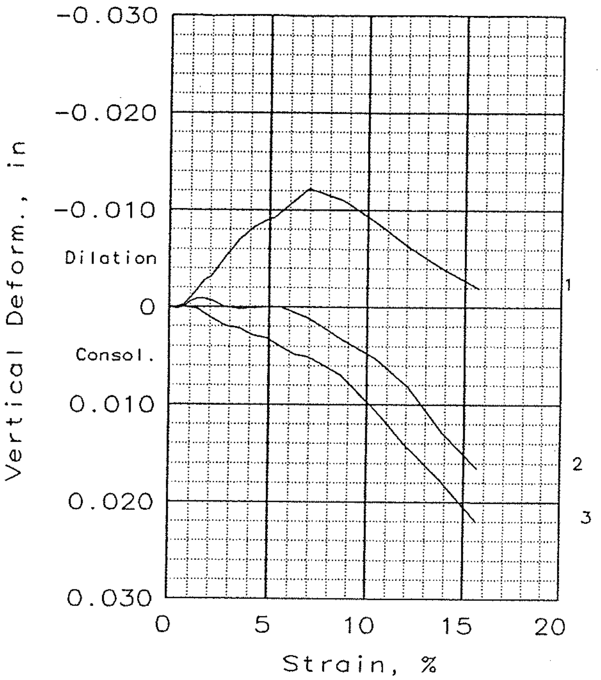
PROJECT: 1697-1

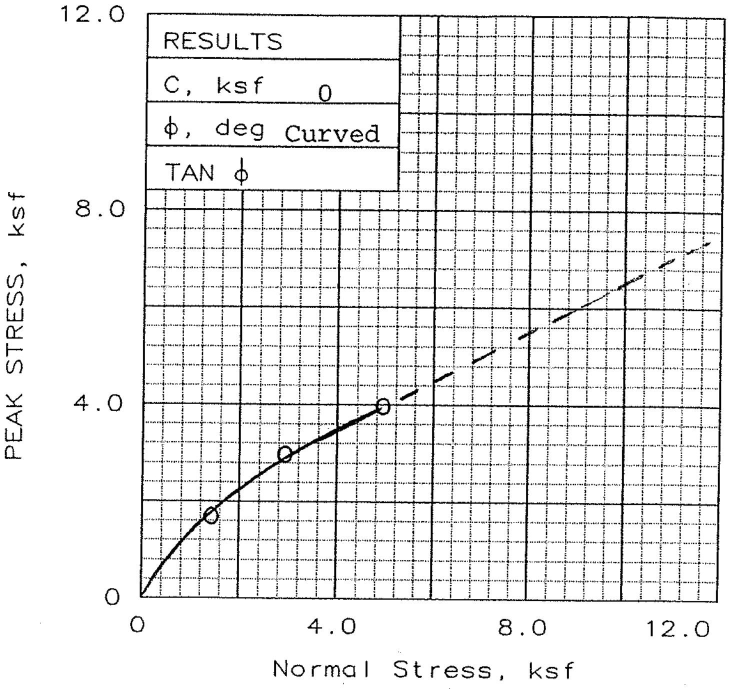
SAMPLE LOCATION: 1)EB15 @ 6' 2)EB16 @ 12' 3)EB17 @ 12.5' 4)EB17 @ 15.5'

PROJ. NO.: 028-975E DATE: 6/22/01

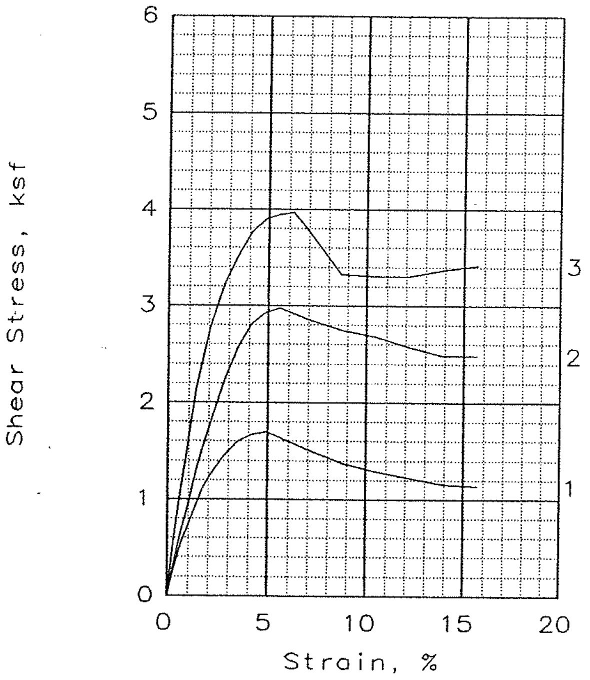
TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY





RESULTS	
C, ksf	0
ϕ, deg	Curved
TAN ϕ	



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	29.5	32.3	28.6
	DRY DENSITY, pcf	91.7	88.2	93.0
	SATURATION, %	94.9	91.9	90.9
	VOID RATIO	0.838	0.983	0.880
	DIAMETER, in	2.87	2.88	2.88
	HEIGHT, in	1.00	1.01	1.03
AT TEST	WATER CONTENT, %	28.5	31.3	27.0
	DRY DENSITY, pcf	93.5	93.0	99.3
	SATURATION, %	95.6	99.6	99.4
	VOID RATIO	0.803	0.880	0.761
	DIAMETER, in	2.87	2.88	2.88
	HEIGHT, in	0.98	0.95	0.96
NORMAL STRESS, ksf		1.50	3.00	5.00
PEAK STRESS, ksf		1.69	2.97	3.97
STRAIN, %		4.9	5.6	6.3
Ultimate Stress, ksf				
STRAIN, %				
Strain rate, %/min		0.07	0.07	0.07

SAMPLE TYPE: undisturbed

DESCRIPTION: dark gray silty SAND w/roots

ASSUMED SPECIFIC GRAVITY= 2.7

REMARKS: Slightly cemented

Note: curved strength envelope

Fig. No.: ______

CLIENT: Lowney

PROJECT: 1697-1

SAMPLE LOCATION: EB12-10A @ 31-34'

PROJ. NO.: 028-957b DATE: 6/01/01

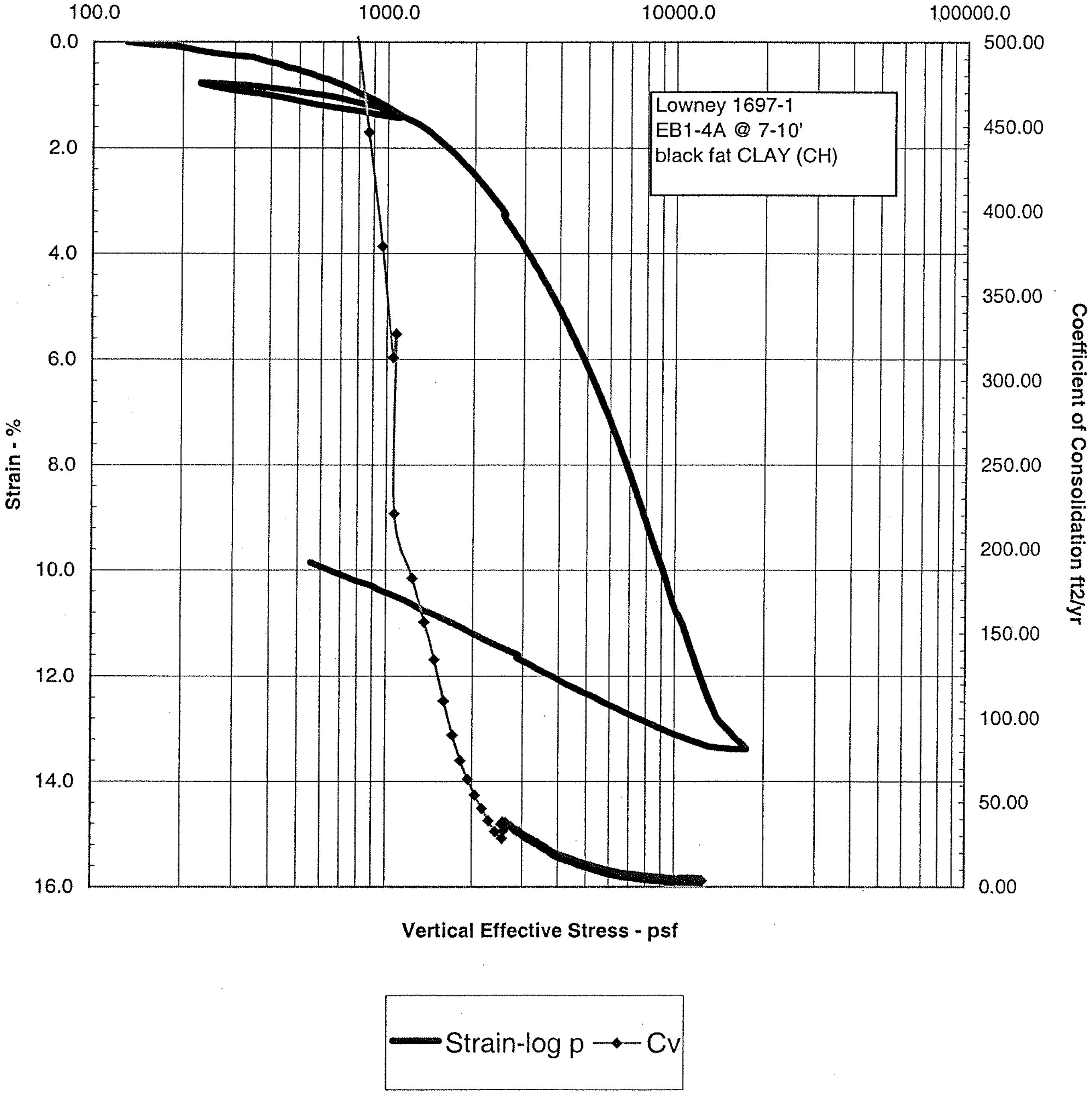
DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY

Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.

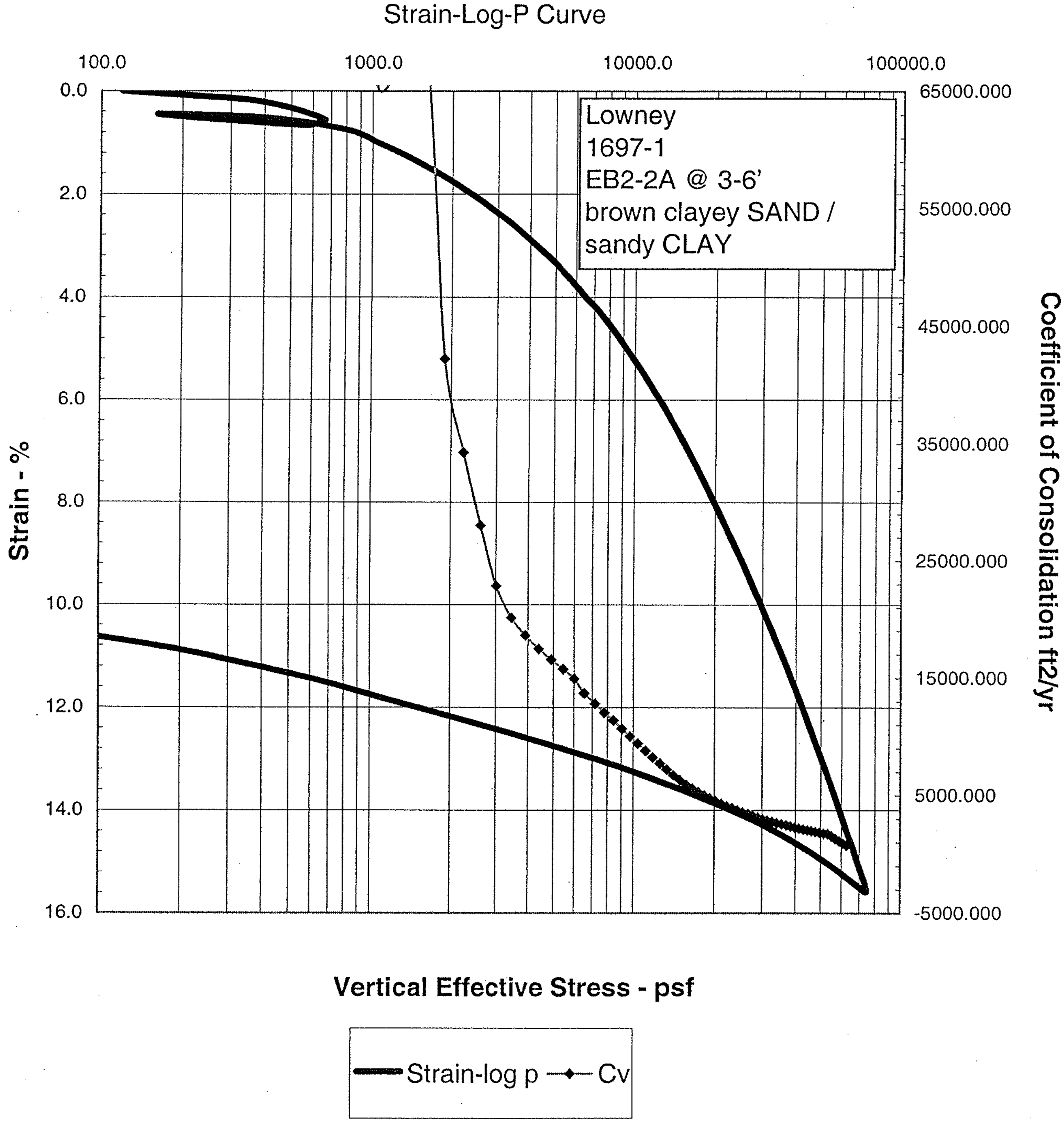
Strain-Log-P Curve



Lowney 1697-1
EB1-4A @ 7-10'
black fat CLAY (CH)

Constant Rate of Strain Consolidation Test

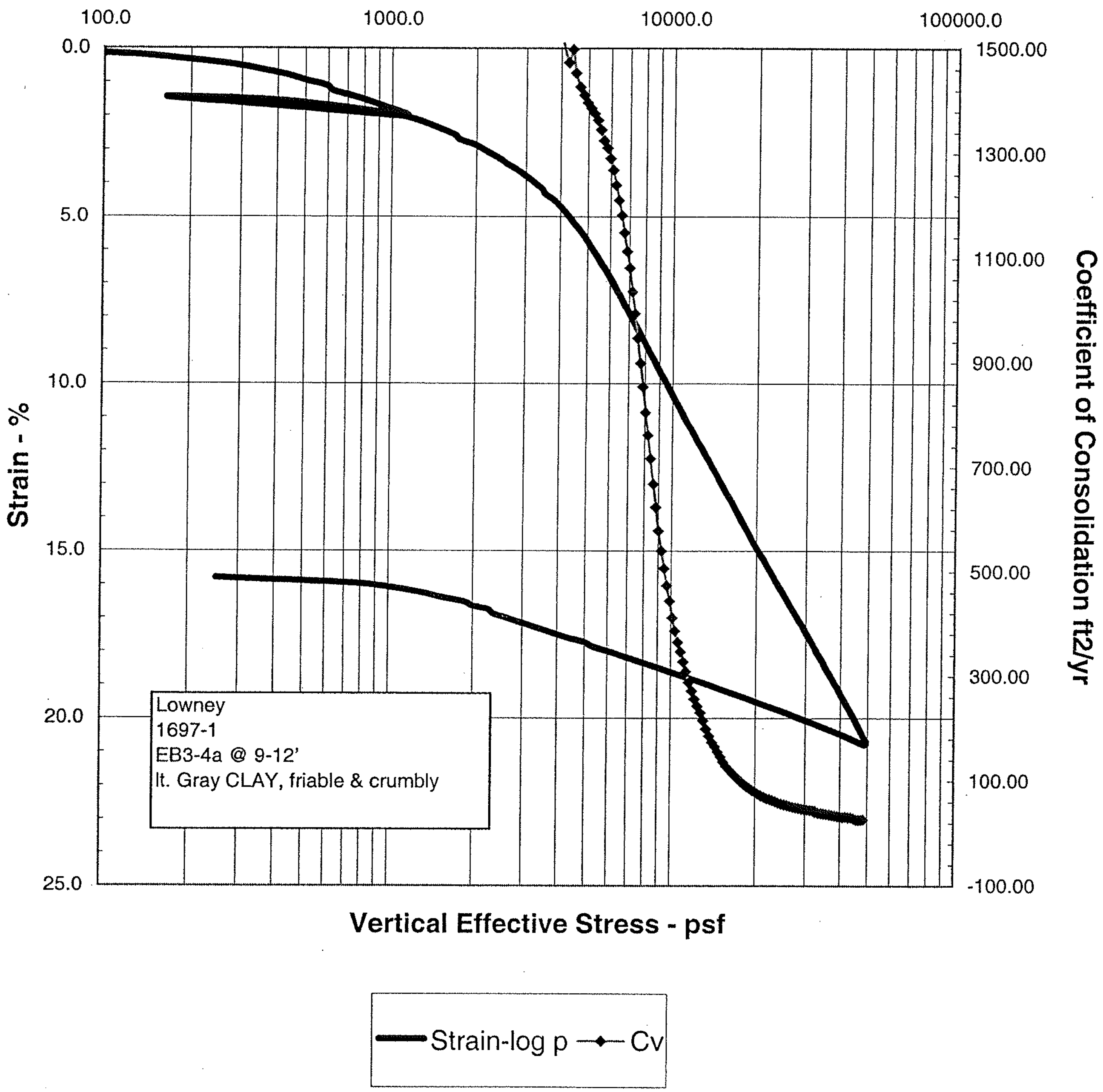
Cooper Testing Labs, Inc.



Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.

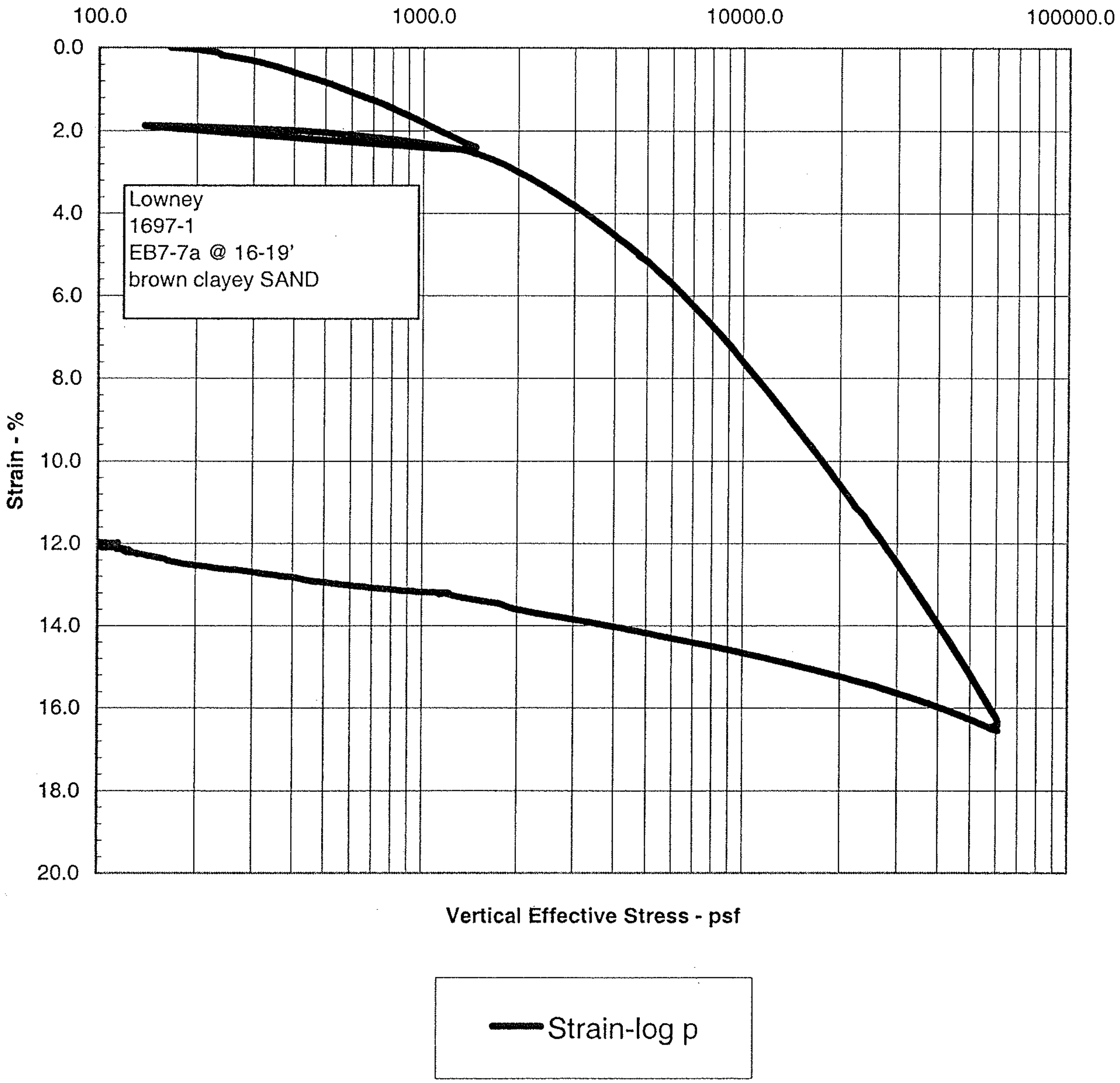
Strain-Log-P Curve



Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.

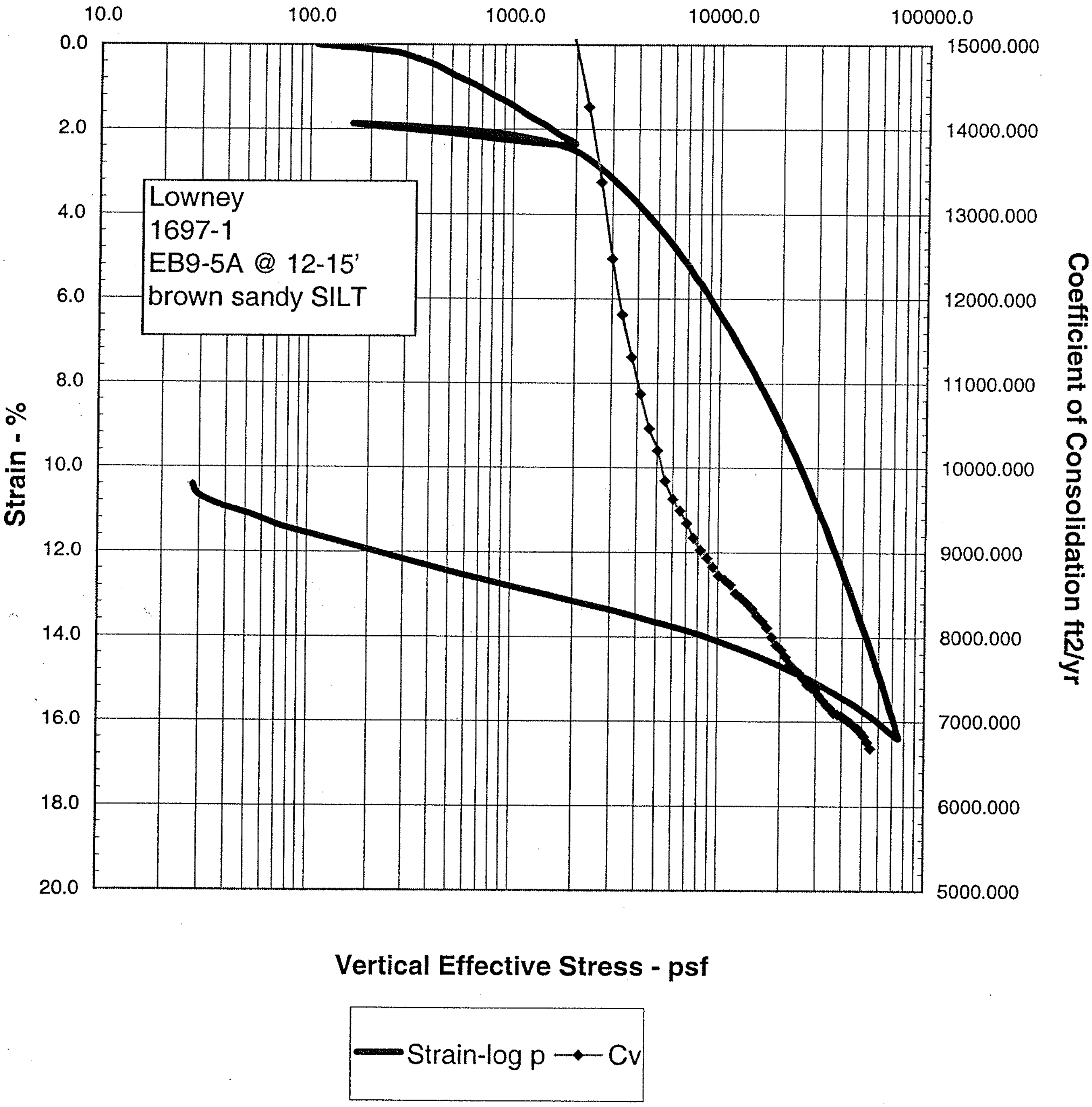
Strain-Log-P Curve



Constant Rate of Strain Consolidation Test

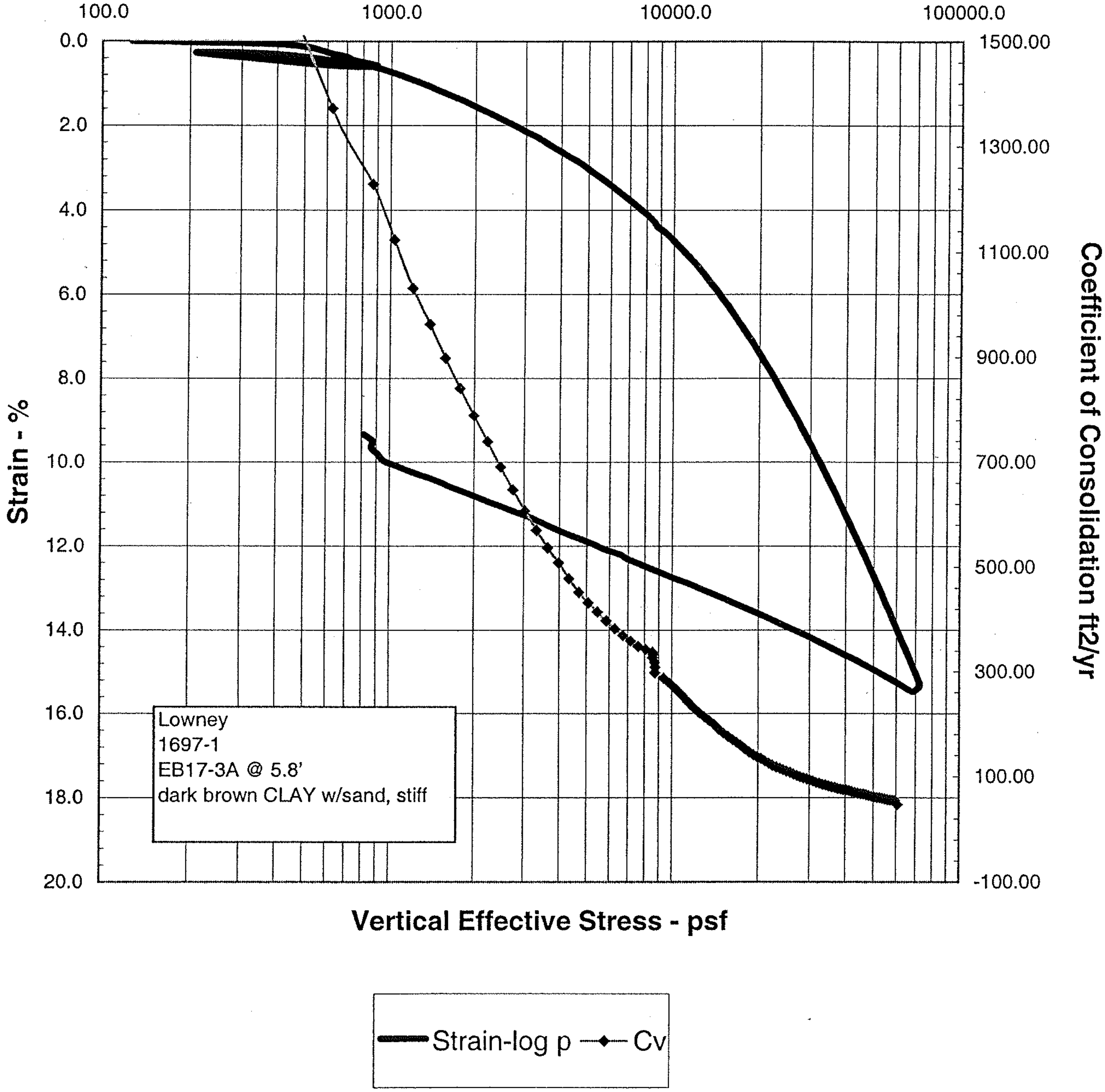
Cooper Testing Labs, Inc.

Strain-Log-P Curve



Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.



Strain-Log-P Curve

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job # Client Project/Location Date	028-962 Lowney 1697-1 6/11/01				
Boring #	EB2-2A	EB2-2A	EB9-5A	EB9-5A	
Depth (ft)	3-6	3-9	12-15	12-15	
Soil Type	brown clayey SAND / sandy CLAY	brown clayey SAND / sandy CLAY	brown silty SAND / sandy SILT	brown clayey SAND / sandy CLAY	
Specific Gravity	2.80 ASSUMED	2.80 ASSUMED	2.80 ASSUMED	2.80 ASSUMED	
Volume Total cc	75.063	69.658	75.063	68.682	
Volume of Solids	45.542	45.536	44.399	44.411	
Volume of Voids	29.521	24.122	30.664	24.271	
Void Ratio	0.648	0.530	0.691	0.547	
Porosity %	39.3%	34.6%	40.9%	35.3%	
Saturation %	95.5%	99.9%	99.7%	99.9%	
Moisture %	22.1%	18.9%	24.6%	19.5%	
Dry Density (pcf)	106.1	114.3	103.4	113.0	

Remarks
Moisture and densities from CRS tests.

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job #	028-957C
Client	Lowney
Project/Location	1697-1
Date	6/21/01

Boring #	EB1-4A	EB1-4A	EB3-4A	EB3-4A	
Depth (ft)	7-10'	7-10	9-12	9-12	
Soil Type	black CLAY (Initial)	same (Final)	light gray CLAY, friable (Initial)	same (Final)	
Specific Gravity	2.70 ASSUMED	2.70 ASSUMED	2.70 ASSUMED	2.70 ASSUMED	
Volume Total cc	75.063	71.610	75.063	69.658	
Volume of Solids	37.336	37.341	40.911	40.979	
Volume of Voids	37.727	34.269	34.152	28.679	
Void Ratio	1.010	0.918	0.835	0.700	
Porosity %	50.3%	47.9%	45.5%	41.2%	
Saturation %	97.0%	100.0%	78.6%	99.9%	
Moisture %	36.3%	34.0%	24.3%	25.9%	
Dry Density (pcf)	83.8	87.9	91.9	99.2	

Remarks

MD data from CRS tests.

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job # Client Project/Location Date	028-945 Lowney 1697-1 5/2/01				
Boring #	EB-7/7A	EB-7/7A			
Depth (ft)	16-19'	16-19			
Soil Type	brown clayey SAND Initial	same final			
Specific Gravity	2.70 ASSUMED	2.70 ASSUMED			
Volume Total cc	75.063	67.181			
Volume of Solids	48.840	48.851			
Volume of Voids	26.223	18.330			
Void Ratio	0.537	0.375			
Porosity %	34.9%	27.3%			
Saturation %	92.0%	99.3%			
Moisture %	18.3%	13.8%			
Dry Density (pcf)	109.7	122.6			

Remarks

M&D from CRS test.

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job #	028-957
Client	Lowney
Project/Location	1697-1
Date	5/30/01

Boring #	EB12-3A				
Depth (ft)	5-6.5				
Soil Type	black CLAY w/sand pockets				
Specific Gravity	2.70 ASSUMED				
Volume Total cc	636.071				
Volume of Solids	388.555				
Volume of Voids	247.516				
Void Ratio	0.637				
Porosity %	38.9%				
Saturation %	69.9%				
Moisture %	16.5%				
Dry Density (pcf)	103.0				

Remarks

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-957c	Boring:	EB-1	Date:	05/23/01
Client:	Lowney	Sample:	4A	By:	DC
Project:	1697-1	Depth:	7-10'		
Soil:	black CLAY				

Sample Pressures:

Cell: 65.4/72.4 Bot. Cap: 59 psi Top Cap: 58 psi Max. Hydraulic Gradient: 14

Elapsed Time (min)		Head, (cm)	K, cm/sec	*B: = 0.97
0	Sigma_3 = 6.9 psi	98.33	Start of Test	Ave. K
152		96.23	2.0 x 10E-7	
314		94.73	1.7 x 10E-7	
389		94.33	1.5 x 10E-7	
440		93.93	1.5 x 10E-7	
0	Sigma_3 = 13.9psi	98.33	Start of Test	2 x 10E-7
7.5		96.33	3.9 x 10E-8	
1035		95.33	3.9 x 10E-8	
1417		94.48	3.5 x 10E-8	
2114		93.13	3.3 x 10E-8	
0		98.33		
2523		92.73	3.0 x 10E-8	3 x 10E-8

Sample Data:	Initial		Final
Height, in	3.00		2.95
Diameter, in	2.87		2.85
Area, in2	6.47		6.36
Volume in3	19.41		18.75
Total Volume, cc	318.03		307.31
Volume Solids, cc	146.19		146.19
Volume Voids, cc	171.85		161.12
Void Ratio	1.18		1.10
Porosity, %	54.03		52.43
Saturation, %	87.93		95.27
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	545.8		548.2
Dry Weight, gm	394.7		394.7
Tare, gm	0.00		0.00
Moisture, %	38.3		38.9
Dry Density, pcf	77.4		80.1

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-961a	Boring:	EB-2	Date:	05/23/01
Client:	Lowney	Sample:	2A	By:	DC
Project:	1697-1	Depth:	3-6'		
Soil:	brown sandy CLAY				

Sample Pressures:

Cell: 85.4/92.4 Bot. Cap: 79 psi Top Cap: 78 psi Max. Hydraulic Gradient: 13

Elapsed Time (min)		Head, (cm)	K, cm/sec	*B: = 0.96
0	Sigma_3 = 6.9psi	98.33	Start of Test	Ave. K
117		95.83	2.9 x 10E-7	
452		88.28	2.9 x 10E-7	
1157		74.83	3.0 x 10E-7	
1447		70.43	3.0 x 10E-7	3 x 10E-7
0	Sigma_3 = 13.9psi	98.33	Start of Test	
174		95.53	2.0 x 10E-7	
856		85.53	2.1 x 10E-7	
1134		81.93	2.2 x 10E-7	
1262		80.28	2.1 x 10E-7	2 x 10E-7

Sample Data:	Initial		Final
Height, in	3.00		2.98
Diameter, in	2.87		2.86
Area, in2	6.47		6.40
Volume in3	19.41		19.08
Total Volume, cc	318.03		312.62
Volume Solids, cc	191.48		191.48
Volume Voids, cc	126.55		121.14
Void Ratio	0.66		0.63
Porosity, %	39.79		38.75
Saturation, %	93.64		95.10
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	635.5		632.2
Dry Weight, gm	517.0		517.0
Tare, gm	0.00		0.00
Moisture, %	22.9		22.3
Dry Density, pcf	101.4		103.2

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-961	Boring:	EB3	Date:	05/23/01
Client:	Lowney	Sample:	6A	By:	DC
Project:	1697-1	Depth:	15-18'		
Soil:	lt brown clayey SAND				

Sample Pressures:

Cell:	49/59psi	Bot. Cap:	39 psi	Top Cap:	39 psi	Max. Hydraulic Gradient:	13

Elapsed Time (min)		Head, (cm)	K, cm/sec	*B = 0.97
0	Sigma_3 = 10.4 psi	28.0	Start of Test	Ave. K
2		24.7	7.7 x 10E-5	
4		22.3	7.0 x 10E-5	
6		20.2	6.7 x 10E-5	
8		18.15	6.6 x 10E-5	
11		15.5	6.6 x 10E-5	7 x 10E-5
0	Sigma_3 = 20.8psi	28.0	Start of Test	
10		17.2	6.4 x 10E-5	
0		28.0	Restart	
1		26.7	6.6 x 10E-5	
2		25.5	6.1 x 10E-5	
4		23.3	6.0 x 10E-5	6 x 10E-5

Sample Data:	Initial	Final
Height, in	3.00	2.96
Diameter, in	2.87	2.85
Area, in2	6.47	6.38
Volume in3	19.41	18.88
Total Volume, cc	318.03	309.44
Volume Solids, cc	214.74	214.74
Volume Voids, cc	103.29	94.70
Void Ratio	0.48	0.44
Porosity, %	32.48	30.60
Saturation, %	85.87	94.41
Specific Gravity	2.70 Assumed	2.70
Wet Weight, gm	668.5	669.2
Dry Weight, gm	579.8	579.8
Tare, gm	0.00	0.00
Moisture, %	15.3	15.4
Dry Density, pcf	113.8	116.9

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-957	Boring:	EB-3	Date:	05/18/01
Client:	Lowney	Sample:	4-A	By:	DC
Project:	1697-1	Depth:	9-12'		
Soil:	lt gray CLAY, friable				

Sample Pressures:

Cell: 55.4/62.4 Bot. Cap: 48.5 psi Top Cap: 48.5 psi Max. Hydraulic Gradient: 8

*B: = 0.95

Elapsed Time (min)		Head, (in)	K, cm/sec	Ave. K
0	Sigma-3 = 6.9 psi	24.0	Start of Test	
0.5		18.6	4.9 x 10E-4	
1		15.0	4.6 x 10E-4	
1.5		12.0	4.5 x 10E-4	
2		9.6	4.4 x 10E-4	
2.5		7.8	4.4 x 10E-4	
3		6.4	4.3 x 10E-4	4 x 10E-4
0	Sigma-3 = 13.9 psi	24.0		
0.5		21.0	2.6 x 10E-4	
1		18.6	2.5 x 10E-4	
3		12	2.3 x 10E-4	
0		24		
1		18.6	2.5 x 10E-4	3 x 10E-4

Sample Data:	Initial		Final
Height, in	3.00		2.93
Diameter, in	2.87		2.86
Area, in2	6.47		6.40
Volume in3	19.41		18.76
Total Volume, cc	318.03		307.38
Volume Solids, cc	160.44		160.44
Volume Voids, cc	157.59		146.93
Void Ratio	0.98		0.92
Porosity, %	49.55		47.80
Saturation, %	78.05		95.21
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	556.2		573.1
Dry Weight, gm	433.2		433.2
Tare, gm	0.00		0.00
Moisture, %	28.4		32.3
Dry Density, pcf	85.0		87.9

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-962	Boring:	EB-9	Date:	05/25/01
Client:	Lowney	Sample:	5A	By:	DC
Project:	1697-1	Depth:	12-15'		
Soil:	brown brown sandy SILT				

Sample Pressures:

Cell: 54.1/58.9 Bot. Cap: 49 psi Top Cap: 48 psi Max. Hydraulic Gradient: 17

Elapsed Time (min)		Head, (cm)	K, cm/sec	*B: = 0.96
0	Sigma_3 = 5.6psi	51.69	Start of Test	Ave. K
2099		13.69	5.7 x 10E-7	
2582		8.99	6.7 x 10E-7	
0		51.69		
102		47.39	8.6 x 10E-7	
0		51.69		
43		50.49	5.7 x 10E-7	7 x 10E-7
0	Sigma_3 = 10.4psi	51.69	Start of Test	
398		41.99	5.2 x 10E-7	
627		37.69	5.3 x 10E-7	
753		35.79	5.2 x 10E-7	
1142		29.29	4.5 x 10E-7	
1929		19.69	5.1 x 10E-7	5 x 10E-7

Sample Data:	Initial		Final
Height, in	3.00		2.81
Diameter, in	2.88		2.90
Area, in2	6.49		6.58
Volume in3	19.48		18.50
Total Volume, cc	319.14		303.10
Volume Solids, cc	190.19		190.19
Volume Voids, cc	128.96		112.92
Void Ratio	0.68		0.59
Porosity, %	40.41		37.25
Saturation, %	98.71		98.74
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	640.8		625.0
Dry Weight, gm	513.5		513.5
Tare, gm	0.00		0.00
Moisture, %	24.8		21.7
Dry Density, pcf	100.4		105.7

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-975a	Boring:	EB11	Date:	06/28/01
Client:	Lowney	Sample:	2A	By:	DC
Project:	1697-1	Depth:	3-6'		
Soil:	brown CLAY				

Sample Pressures:

Cell: 56.5/52.5 Bot. Cap: 49.5 psi Top Cap: 47.5 psi Max. Hydraulic Gradient: 13

*B:= 1.0

Elapsed Time (min)		Head, (cm)	K, cm/sec	Ave. K
0	Sigma3 = 4 psi	168.67	Start of Test	
		165.17	2.1 x 10E-8	
		163.83	2.0 x 10E-8	
		168.67		
		165.48	2.4 x 10E-8	
		164.07	2.3 x 10E-8	2 x 10E-8
	Sigma3 = 8 psi	98.33		
		97.53	1.8 x 10E-8	
		97.33	1.2 x 10E-8	
		96.93	9.4 x 10E-9	
		96.33	1.0 x 10E-8	1 x 10E-8

*Staged permeability test using two confining pressures.

Sample Data:	Initial		Final
Height, in	3.00		3.01
Diameter, in	2.86		2.86
Area, in2	6.42		6.44
Volume in3	19.27		19.38
Total Volume, cc	315.82		317.54
Volume Solids, cc	176.84		176.84
Volume Voids, cc	138.99		140.70
Void Ratio	0.79		0.80
Porosity, %	44.01		44.31
Saturation, %	92.67		98.36
Specific Gravity	2.75	Assumed	2.75
Wet Weight, gm	615.1		624.7
Dry Weight, gm	486.3		486.3
Tare, gm	0.00		0.00
Moisture, %	26.5		28.5
Dry Density, pcf	96.1		95.6

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-975	Boring:	EB-15	Date:	06/27/01
Client:	Lowney	Sample:	5A	By:	DC
Project:	1697-1	Depth:	3.5-6.5'		
Soil:	brown CLAY w/sand				

Sample Pressures: Max. Hydraulic
Cell: 51.5/55 Bot. Cap: 49 psi Top Cap: 47 psi Gradient: 26
*B:=1.0

Elapsed Time (min)		Head, (cm)	K, cm/sec	Ave. K
0	Sigma3 = 3.5psi	79.38	Start of Test	
309		78.58	3.1 x 10E-8	
441		78.38	2.7 x 10E-8	
1355		77.08	2.2 x 10E-8	
1830		76.53	1.9 x 10E-8	
0	Sigma3 = 7psi	79.38	Restart	2 x 10E-8
655		78.38	1.6 x 10E-8	
1345		77.68	1.5 x 10E-8	
1809		74.48	2.0 x 10E-8	
3209		73.88	1.7 x 10E-8	
4224		73.63	1.6 x 10E-8	2 x 10E-8

*Staged permeability test using two confining pressures.

Sample Data:	Initial		Final
Height, in	3.00		2.99
Diameter, in	2.85		2.85
Area, in2	6.39		6.38
Volume in3	19.17		19.07
Total Volume, cc	314.06		312.57
Volume Solids, cc	173.25		173.25
Volume Voids, cc	140.81		139.32
Void Ratio	0.81		0.80
Porosity, %	44.84		44.57
Saturation, %	92.40		98.48
Specific Gravity	2.80	Assumed	2.80
Wet Weight, gm	615.2		622.3
Dry Weight, gm	485.1		485.1
Tare, gm	0.00		0.00
Moisture, %	26.8		28.3
Dry Density, pcf	96.4		96.8

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

Hydraulic Conductivity
ASTM D 5084
Cooper Testing Lab, Inc.

Job No:	028-957a	Boring:	EB-17	Date:	05/23/01
Client:	Lowney	Sample:	3A	By:	DC
Project:	1697-1	Depth:	5-8'		
Soil:	dark brown CLAY w/sand				

Sample Pressures:

Cell: 44.1/52.4 Bot. Cap: 39 psi Top Cap: 38 psi

Max. Hydraulic Gradient: 17

*B:= 0.99

Elapsed Time (min)		Head, (in)	K, cm/sec	Ave. K
0	Sigma_3 = 5.6 psi	51.69	Start of Test	
2431		40.69	9.7 x 10E-8	
3179		38.29	8.7 x 10E-8	
3568		37.19	8.7 x 10E-8	
3866		36.39	9.1 x 10E-8	9 x 10E-8
0	Sigma_3 = 13.9psi	51.69	Start of Test	
678		50.49	2.3 x 10E-8	
958		49.99	2.5 x 10E-8	
1434		48.89	2.2 x 10E-8	
2776		46.79	2.1 x 10E-8	
3077		46.44	2.9 x 10E-8	2 x 10E-8

Sample Data:	Initial	Final
Height, in	3.00	2.99
Diameter, in	2.87	2.87
Area, in2	6.47	6.47
Volume in3	19.41	19.34
Total Volume, cc	318.03	316.97
Volume Solids, cc	182.22	182.22
Volume Voids, cc	135.81	134.75
Void Ratio	0.75	0.74
Porosity, %	42.70	42.51
Saturation, %	87.69	95.36
Specific Gravity	2.70 Assumed	2.70
Wet Weight, gm	611.1	620.5
Dry Weight, gm	492.0	492.0
Tare, gm	0.00	0.00
Moisture, %	24.2	26.1
Dry Density, pcf	96.5	96.9

Remarks: *B=Delta Pore Press/Delta Cell Press (indication of saturation).

COOPER TESTING LABORATORIES

R-VALUE TEST

JOB #: 028-974B
DATE: 6/08/2001
CLIENT: Lowney / 1697-1 / Berryessa
SAMPLE #: EB-15 @ 1.0/ Bucket # 5
SOIL TYPE: brown CLAY with sand and gravel

DISH WEIGHT: 54.1
WET: 243.6
DRY: 222.2
INITIAL MOSITURE: 0.1273

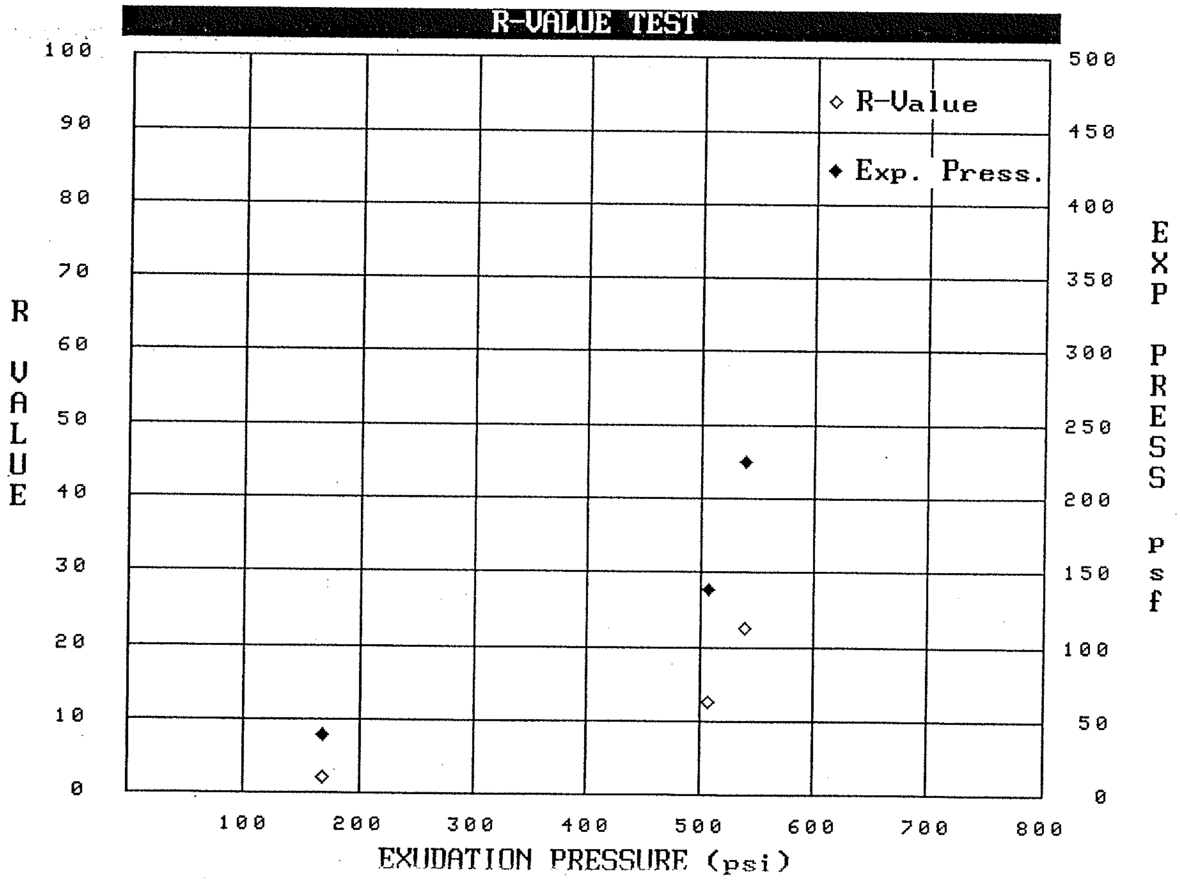
SPECIMEN	A	B	C	D
EXUDATION PRESSURE (psi)	541	> 509	> 170	0
PREPARED WEIGHT (gm)	1200	1200	1200	1200
FINAL WATER ADDED (gm)	60	85	140	0
WEIGHT, SOIL & MOLD (gm)	3182	3135	3141	0
WEIGHT, MOLD (gm)	2102	2104	2108	0
HEIGHT (in)	2.52	2.45	2.60	0.00
MOISTURE CONTENT (%)	18.4	20.7	25.9	0.0
DRY DENSITY (pcf)	109.6	105.6	95.6	0.0
EXPANSION DIAL	52	32	9	0
EXPANSION PRESSURE (psf)	224	138	39	0
STABILOMETER @ 2000 lb	128	138	154	0
TURNS DISPLACEMENT	2.18	2.55	3.59	0.00
R-VALUE	22	14	3	0
R-VALUE (corrected)	22	12	2	0

VALUES AT 300 EXUDATION

R-VALUE: < 5
EXP. PRESSURE: 0

REMARKS

Soil extruded from under mold during exudation giving a false pressure. Per Caltrans the test was discontinued and reported as less than 5 R-value.



COOPER TESTING LABORATORIES

R-VALUE TEST

JOB #: 028-974
DATE: 6/08/2001
CLIENT: Lowney / 1697-1 / Berryessa
SAMPLE #: EB-15 @ 4' / Bucket #22
SOIL TYPE: black CLAY with sand and gravel

DISH WEIGHT: 33.0
WET: 241.4
DRY: 217.7
INITIAL MOSITURE: 0.1283

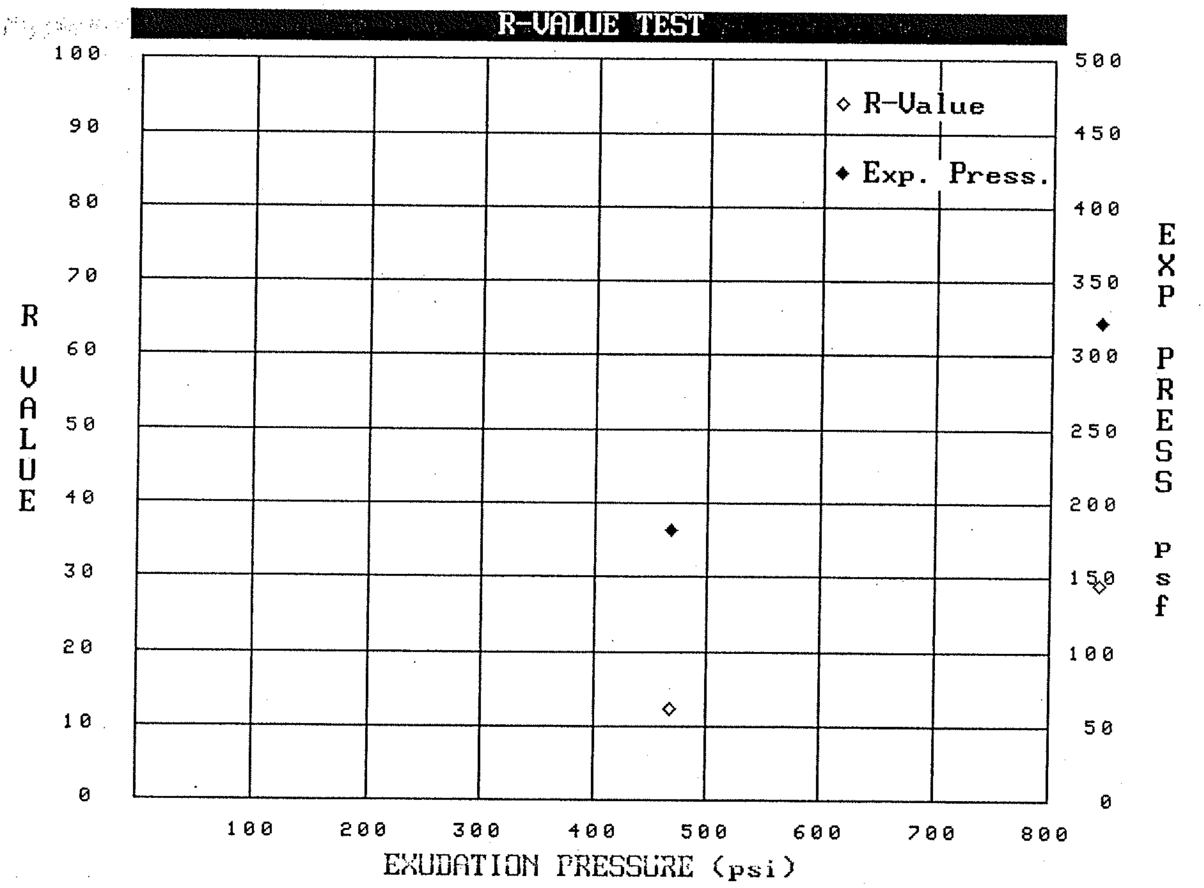
SPECIMEN	A	B	C	D
EXUDATION PRESSURE (psi)	846	470	0	0
PREPARED WEIGHT (gm)	1200	1200	1200	1200
FINAL WATER ADDED (gm)	80	130	0	0
WEIGHT, SOIL & MOLD (gm)	3118	3031	0	0
WEIGHT, MOLD (gm)	2123	2081	0	0
HEIGHT (in)	2.44	2.42	0.00	0.00
MOISTURE CONTENT (%)	20.4	25.1	0.0	0.0
DRY DENSITY (pcf)	102.6	95.1	0.0	0.0
EXPANSION DIAL	75	42	0	0
EXPANSION PRESSURE (psf)	323	181	0	0
STABILOMETER @ 2000 lb	115	140	0	0
TURNS DISPLACEMENT	2.23	2.38	0.00	0.00
R-VALUE	30	13	0	0
R-VALUE (corrected)	29	12	0	0

VALUES AT 300 EXUDATION

R-VALUE: <5
EXP. PRESSURE: 0

REMARKS

Soil extruded from under mold during exudation giving a false pressure. Per Caltrans the test was discontinued and reported as less than 5 R-value



COOPER TESTING LABORATORIES

R-VALUE TEST

JOB #: 028-974A
DATE: 6/08/2001
CLIENT: Lowney / 1697-1 / Berryessa
SAMPLE #: EB-4 @ 1.5 / Bucket #30
SOIL TYPE: gray CLAY with sand and gravel

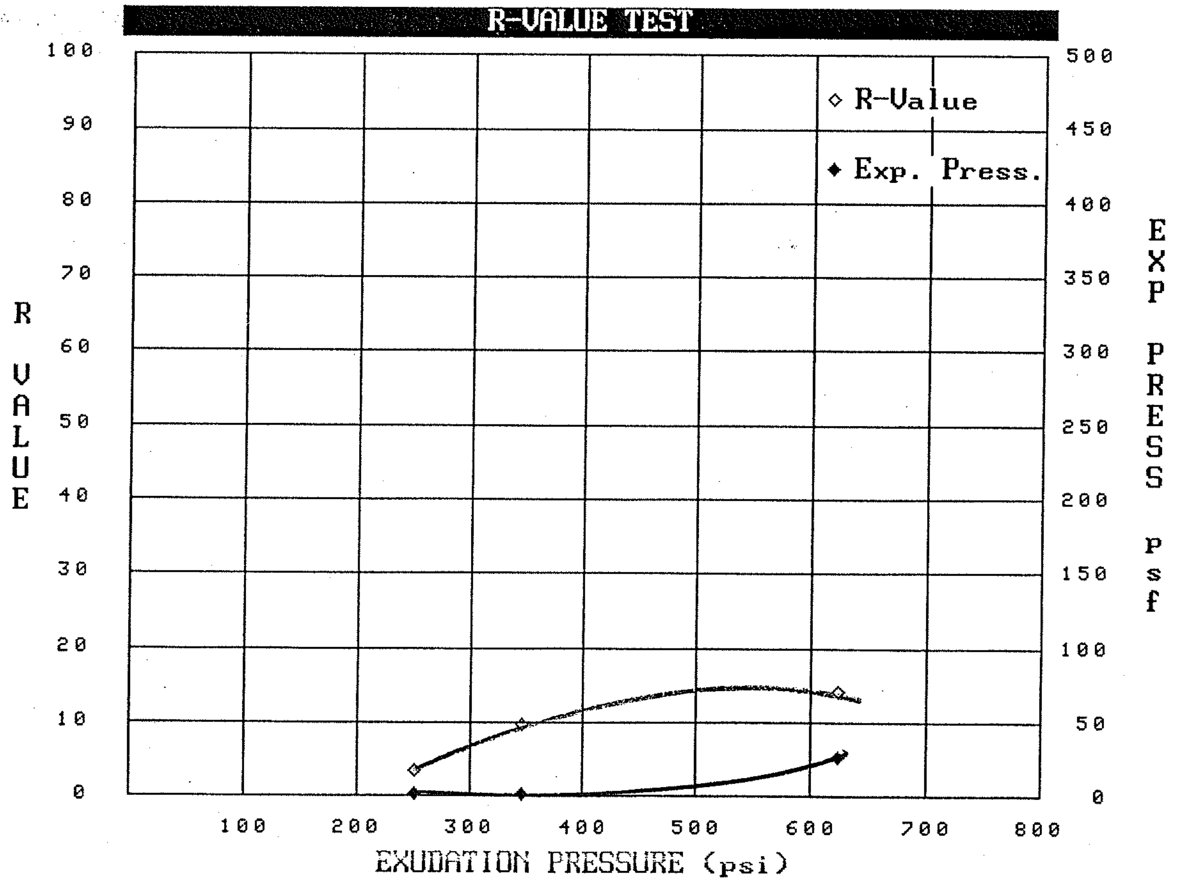
DISH WEIGHT: 43.1
WET: 270.8
DRY: 246.6
INITIAL MOSITURE: 0.1189

SPECIMEN	A	B	C	D
EXUDATION PRESSURE (psi)	348	254	626	0
PREPARED WEIGHT (gm)	1200	1200	1200	1200
FINAL WATER ADDED (gm)	66	108	40	0
WEIGHT, SOIL & MOLD (gm)	3130	3100	3138	0
WEIGHT, MOLD (gm)	2117	2095	2107	0
HEIGHT (in)	2.40	2.48	2.40	0.00
MOISTURE CONTENT (%)	18.0	22.0	15.6	0.0
DRY DENSITY (pcf)	108.3	100.6	112.5	0.0
EXPANSION DIAL	0	0	6	0
EXPANSION PRESSURE (psf)	0	0	26	0
STABILOMETER @ 2000 lb	142	153	137	0
TURNS DISPLACEMENT	2.66	3.38	2.34	0.00
R-VALUE	11	3	15	0
R-VALUE (corrected)	9	3	14	0

VALUES AT 300 EXUDATION

R-VALUE: 6
EXP. PRESSURE: 0

REMARKS



INSTRUCTIONS FOR COMPLETING THE LEVEE/FLOODWALL SYSTEM ANALYSES FORM (FORM 8)

The purpose of this form is to assure that the levee or floodwall is designed and/or constructed to provide protection from the 1% annual chance flood, in full compliance with 44 CFR Ch. 1, Section 65.10 of the NFIP regulations, before reflecting its effects on an NFIP map. A complete engineering analysis must be submitted in support of each section of this form. In addition, a vicinity map along with a complete set of flood profile sheets, plan sheets, and layout detail sheets must be submitted. These sheets must be numbered, and an index must be provided that clearly identifies those sheets specifically relating to the levee or floodwall in question.

Reach to be Revised

Indicate the extent of the levee/floodwall system.

Levee/Floodwall System Elements

Indicate all the levee/floodwall system elements that apply and provide engineering drawings certified by a registered professional engineer.

Freeboard

Riverine levees must provide a minimum freeboard of three feet above the 1% annual chance water-surface elevation. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. If exceptions to the minimum freeboard requirements are requested, attach documentation addressing Subparagraph 65.10(b)(ii) if the NFIP regulations.

Sediment Transport Consideration

Provide any necessary information if there is evidence that sediment transport will affect the 1% annual chance water-surface elevations.

Closures

All openings must be provided with closure devices that are structural parts of the system during operation and design.

Embankment Protection

The embankment protection analysis must demonstrate that no appreciable erosion of the levee embankment can be expected during the 1% annual chance flood, as a result of either current or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability. Factors to addressed include, but are not limited to: expected flow velocities; expected wind and wave action; ice loading; impact of debris; slope protection techniques; duration of flooding at various stages and velocities; embankment and foundation materials; levee alignment; bends; transitions; and levee side slopes.

Attach engineering analysis to support the construction plans. Submit all backup information used in the analysis.

Embankment and Foundation Stability

This analysis must evaluate expected seepage during loading conditions associated with the 1% annual chance flood and shall demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment and foundation stability. An alternative analysis described in the USACE manual, "Design and Construction of Levees" (EM 1110-2-1913, Chapter 6, Section II), may be used. The factors that must be addressed in the analysis include: depth of flooding, duration of flooding, embankment geometry and length of seepage path at critical locations, others design factors (such as drainage layers), and others design factors affecting embankment and foundation stability (such as berms). Submit all backup information used in the analysis.

Floodwall and Foundation Stability

See above embankment and foundation stability discussion.

Settlement

The settlement analysis must assess the potential and magnitude of future losses of freeboard and must demonstrate that the minimum freeboard requirements will be maintained. The analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. In addition, detailed settlement analysis using procedures such as those described in the USACE manual, "Soil Mechanics Design-Settlement Analysis" (EM 1100-2-1904) must be submitted. Submit all backup information used in the analysis.

Interior Drainage

In accordance with Subparagraph 65.10(b)(6) of the NFIP regulations, the interior drainage analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities for evacuating interior floodwaters. The analysis must identify the extent of the flooded area, and the water-surface elevation(s) of the 1% annual chance flood if the average depth is greater than one foot. This information is to show on a certified topographic workmap. All back-up information must be submitted.

Other Design Criteria

Address any other criteria that may be a problem and attach any supporting documentation.

Operational Plan and Criteria

For a levee system to be recognized by FEMA, the operational criteria must be as described in Subparagraph 65.10(c) of the NFIP regulations.

6. EMBANKMENT PROTECTION

1. The maximum levee slope landside is: ____________________

2. The maximum levee slope floodside is: ____________________

3. The range of 100-year (base) riverine flood velocities along the levee?__________ (min.) to ____________ (max.)

4. Embankment material is protected by (describe the kind): ____________________

5. Riprap Design Parameters: (Include references) ☐ Velocity ☐ Tractive stress

Reach	Sideslope	Flow Depth	Velocity	Curve or Straight	Stone Riprap			Depth of Toedown
					D_{100}	D_{50}	Thickness	
Sta to								
Sta to								
Sta to								
Sta to								
Sta to								
Sta to								

(Extend table on an added sheet as needed and reference)

6. Is a bedding/filter analysis and design attached? ☐ Yes ☐ No

7. Describe the analysis used for other kinds of protection used (include copies of the design analysis):

____

Note: Attach engineering analysis to support construction plans.

7. EMBANKMENT AND FOUNDATION STABILITY

1. Identify locations and describe the basis for selection of critical location for analysis:____________________

____

☐ Overall height: Sta _________, height______________ ft.

☐ Limiting foundation soil strength:

Sta _________, depth _________ to ___________

Strength Ø = ________ degrees, c =__________ psf

slope: SS =_________ (h) to _________ (v)

(Repeat as needed on an added sheet for additional locations)

2. Specify the embankment stability analysis methodology used (e.g., circular arc, sliding block, infinite slope, etc.):

3. Summary of stability analysis results:

Case	Loading Conditions	Critical Safety Factor	Criteria (Min.)
I	End of construction		1.3
II	Sudden drawdown		1.0
III	Critical flood stage		1.4
IV	Steady seepage at flood stage		1.4
VI	Earthquake (Case I)		1.0

(Reference: U.S. Army Corps of Engineers (USACE) EM-1110-2-1913 Table 6-1)

4. Was a seepage analysis for the embankment performed? ☐ Yes ☐ No

If Yes, describe methodology used: ______________________________

____

5. Was a seepage analysis for the foundation performed: ☐ Yes ☐ No

6. Were uplift pressures at the embankment landside toe checked? ☐ Yes ☐ No

7. Were seepage exit gradients checked for piping potential? ☐ Yes ☐ No

8. The duration of 100-year (base) flood hydrograph against the embankment is ________ hours.

Note: **Attach engineering analysis to support construction plans.**

8. FLOODWALL AND FOUNDATION STABILITY

1. Describe analysis submittal based on Code:

   ☐ UBC (1988) or ☐ Other (specify):

2. Stability analysis submitted provides for:

   ☐ Overturning ☐ Sliding; If not, explain: ____________________

   ____

3. Loading included in the analyses were:

   ☐ Lateral earth @ P_A = ________ psf; P_p = ____________ psf

   ☐ Surcharge-Slope @ ________, ☐ surface ________ psf

   ☐ Wind @ P_w = ____________ psf

   ☐ Seepage (Uplift); __________ ☐ Earthquake @ P_{eq} = __________ %g

   ☐ 100-year significant wave height____________ ft.

   ☐ 100-year significant wave period____________ sec.

4. Summary of Stability Analysis Results: Factors of Safety. Itemize for each range in site layout dimension and loading condition limitation for each respective reach.

Loading Condition	Criteria (Min)		Sta	To	Sta	To
	Overturn	Sliding	Overturn	Sliding	Overturn	Sliding
Dead & Wind	1.5	1.5				
Dead & Soil	1.5	1.5				
Dead, Soil, Flood, & Impact	1.5	1.5				
Dead, Soil, & Seismic	1.3	1.3				

(Ref: FEMA 114 Sept 1986; USACE EM 1110-2-2502)

(Note: Extend table on an added sheet as needed and reference)

5. Foundation bearing strength for each soil type:

Bearing Pressure	Sustained Load (psf)	Short Term Load (psf)
Computed design maximum		
Maximum allowable		

6. Foundation scour protection ☐ is, ☐ is not provided. Describe if provided:

   ____

   ____

   ____

Note: Attach engineering analysis to support construction plans.

9. SETTLEMENT

1. Has anticipated potential settlement been determined and incorporated into the specified construction elevations to maintain the established freeboard margin? ☐ Yes ☐ No

2. The computed range of settlement is ________ ft. to _________ ft.

3. Settlement of the levee crest is determined to be primarily from:

☐ Foundation consolidation
☐ Embankment compression
☐ Other (describe):

4. Differential settlement of floodwalls

☐ has ☐ has not been accommodated in the structural design and construction.

Note: Attach engineering analysis to support construction plans.

10. INTERIOR DRAINAGE

1. Specify size of each interior watershed

Draining to pressure conduit: _______________

Draining to ponding area: _______________

2. Relationships Established

Ponding elevation vs. storage ☐ Yes ☐ No
Ponding elevation vs. gravity flow ☐ Yes ☐ No
Differential head vs. gravity flow ☐ Yes ☐ No

3. The river flow duration curve is enclosed ☐ Yes ☐ No

4. Specify the discharge capacity of the head pressure conduit: _______________

5. Which Flooding Conditions Were Analyzed?

- Gravity flow (Interior Watershed) ☐ Yes ☐ No
- Common storm (River Watershed) ☐ Yes ☐ No
- Historical ponding probability ☐ Yes ☐ No
- Coastal wave overtopping ☐ Yes ☐ No

If No, explain why not: ____

____

____

6. Interior drainage has been analyzed based on joint probability of interior and exterior flooding and the capacities of pumping and outlet facilities to provide the established level of flood protection. ☐ Yes ☐ No

If No, explain why not: ____

____

7. The rate of seepage through the levee system for the 100-year (base) flood is_____________ cfs

10. INTERIOR DRAINAGE (Cont'd)

8. The length of levee system used to drive this seepage rate in item 7:____________ ft.

9. Will a pumping plant(s) be used for interior drainage? ☐ Yes ☐ No

If Yes, include the number of pumping plants: ____________
For each pumping plant, list:

	Plant #1	Plant #2
The number of pumps		
The ponding storage capacity		
The maximum pumping rate		
The maximum pumping head		
The pumping starting elevation		
The pumping stopping elevation		
Is the discharge facility protected?		
Is there a flood warning plan?		
How much time is available between warning and flooding?		

Will the operations be automatic? ☐ Yes ☐ No
If the pumps are electric, are there backup power sources? ☐ Yes ☐ No

(Reference: U.S. Army Corps of Engineers EM-1110-2-3101, 3102, 3103, 3104, and 3105)

Note: Include a copy of supporting documentation of data and analysis. Provide a map showing the flooded area and maximum ponding elevations for all interior watersheds that result in flooding.

11. OTHER DESIGN CRITERIA

1. The following items have been addressed as stated:

Liquefaction ☐ is ☐ is not a problem
Hydrocompaction ☐ is ☐ is not a problem
Heave differential movement due to soils of high shrink/swell ☐ is ☐ is not a problem

2. For each of these problems, state the basic facts and corrective action taken:

____

____

____

3. If the levee/floodwall is new or enlarged, will the structure adversely impact flood levels and/or flow velocities floodside of the structure? ☐ Yes ☐ No

Note: Attach supporting documentation

Chapter 6
Slope Design and Settlement

Section I
Embankment Stability

6-1. Embankment Geometry

a. Slopes. For levees of significant height or when there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis. Low levees and levees to be built of good material resting on proven foundations may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of levee slopes.

(1) Type of construction. Fully compacted levees generally enable the use of steeper slopes than those of levees constructed by semicompacted or hydraulic means. In fact, space limitations in urban areas often dictate minimum levee sections requiring select material and proper compaction to obtain a stable section.

(2) Ease of construction. A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers.

(3) Maintenance. A 1V on 3H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment and walked on during inspections.

(4) Seepage. For sand levees, a 1V on 5H landside slope is considered flat enough to prevent damage from seepage exiting on the landside slope.

(5) Slope protection. Riverside slopes flatter than those required for stability may have to be specified to provide protection from damage by wave action.

b. Final Levee Grade. In the past, freeboard was used to account for hydraulic, geotechnical, construction, operation and maintenance uncertainties. The term and concept of freeboard to account for these uncertainties is no longer used in the design of levee projects. The risk-based analysis directly accounts for hydraulic uncertainties and establishes a nominal top of protection. Deterministic analysis using physical properties of the foundation and embankment materials should be used to set the final levee grade to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances.

c. Crown width. The width of the levee crown depends primarily on roadway requirements and future emergency needs. To provide access for normal maintenance operations and floodfighting operations, minimum widths of 3.05 to 3.66 m (10 to 12 ft) are commonly used with wider turnaround areas provided at specified intervals; these widths are about the minimum feasible for construction using modern heavy earthmoving equipment and should always be used for safety concerns. Where the levee crown is to be used as a higher class road, its width is usually established by the responsible agency.

6-2. Standard Levee Sections and Minimum Levee Section

a. Many districts have established standard levee-sections for particular levee systems, which have proven satisfactory over the years for the general stream regime, foundation conditions prevailing in those areas, and for soils available for levee construction. For a given levee system, several different standard sections may be established depending on the type of construction to be used (compacted, semicompacted, uncompacted, or hydraulic fill). The use of standard sections is generally limited to levees of moderate height (say less than 7.62 m (25 ft)) in reaches where there are no serious underseepage problems, weak foundation soils, or undesirable borrow materials (very wet or very organic). In many cases the standard levee section has more than the minimum allowable factor of safety relative to slope stability, its slopes being established primarily on the basis of construction and maintenance considerations. Where high levees or levees on foundations presenting special underseepage or stability problems are to be built, the uppermost riverside and landside slopes of the levee are often the same as those of the standard section, with the lower slopes flattened or stability berms provided as needed.

b. The adoption of standard levee sections does not imply that stability and underseepage analyses are not made. However, when borings for a new levee clearly demonstrate foundation and borrow conditions similar to those at existing levees, such analyses may be very simple and made only to the extent necessary to demonstrate unquestioned levee stability. In addition to being used in levee design, the standard levee sections are applicable to initial cost estimate, emergency and maintenance repairs.

c. The minimum levee section shall have a crown width of at least 3.05 m (10 ft) and a side slope flatter than or equal to 1V on 2H, regardless of the levee height or the possibly less requirements indicated in the results of stability and seepage analyses. The required dimensions of the minimum levee section is to provide an access road for flood-fighting, maintenance, inspection and for general safety conditions.

6-3. Effects of Fill Characteristics and Compaction

a. Compacted fills. The types of compaction, water content control, and fill materials govern the steepness of levee slopes from the stability aspect if foundations have adequate strength. Where foundations are weak and compressible, high quality fill construction is not justified, since these foundations can support only levees with flat slopes. In such cases uncompacted or semicompacted fill, as defined in paragraph 1-5, is appropriate. Semicompacted fill is also used where fine-grained borrow soils are considerably wet of optimum or in construction of very low levees where other considerations dictate flatter levee slopes than needed for stability. Uncompacted fill is generally used where the only available borrow is very wet and frequently has high organic content and where rainfall is very high during the construction season. When foundations have adequate strength and where space is limited in urban areas both with respect to quantity of borrow and levee geometry, compacted levee fill construction by earth dam procedures is frequently selected. This involves the use of select material, water content control, and compaction procedures as described in paragraph 1-5.

b. Hydraulic Fill. Hydraulic fill consists mostly of pervious sands built with one or two end-discharge or bottom-discharging pipes. Tracked or rubber-tired dozers or front-end loaders are used to move the sand to shape the embankment slopes. Because a levee constructed of hydraulic fill would be very pervious and have a low density, it would require a large levee footprint and would be susceptible to soil liquefaction. Hydraulic fill would also quickly erode upon overtopping or where an impervious covering was penetrated. For these reasons, hydraulic fill may be used for stability berms, pit fills and seepage berms but shall not normally be used in constructing levee embankments. However, hydraulic fill may be used for levees protecting agricultural areas whose failure would not endanger human life and for zoned embankments that include impervious seepage barriers.

Section II
Stability Analyses

6-4. Methods of Analysis

The principal methods used to analyze levee embankments for stability against shear failure assume either (a) a sliding surface having the shape of a circular arc within the foundation and/or the embankment or (b) a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface. Various methods of analysis are described in EM 1110-2-1902, and can be chosen for use where determined appropriate by the designer. Computer programs are available for these analyses, with the various loading cases described in EM 1110-2-1902, so the effort of making such analyses is greatly reduced, and primary attention can be devoted to the more important problems of defining the shear strengths, unit weights, geometry, and limits of possible sliding surfaces.

6-5. Conditions Requiring Analysis

The various loading conditions to which a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; Case III, steady seepage from full flood stage, fully developed phreatic surface; Case IV, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given. For more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case refer to EM 1110-2-1902.

a. Case I - End of construction. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

b. Case II - Sudden drawdown. This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. For the selection of the shear strengths see Table 6-1a.

c. Case III - Steady seepage from full flood stage (fully developed phreatic surface). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability. Design shear strengths should be based on Table 6-1a.

d. Case IV - Earthquake. Earthquake loadings are not normally considered in analyzing the stability of levees because of the low probability of earthquake coinciding with periods of high water. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required.

Table 6-1a
Summary of Design Conditions

Analysis Condition	Shear Strength[a]	Pore Water Pressure
During and End-of-Construction	Free draining soils - use effective stresses	Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
	Low permeability soils - use undrained strengths and total stresses[b]	Low permeability soils - Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady State Seepage Conditions	Use effective stresses. Residual strengths should be used where previous shear deformation or sliding has occurred.	Estimated from field measurements of pore water pressures, hydrostatic pressure computations for no flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils - use effective stresses	Free draining soils - First stage computations (before drawdown) - steady-state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.
	Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.	Low permeability soils - First stage computations - steady-state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained strengths are used pore water pressures are set to zero.

[a] Effective stress parameters can be obtained from consolidated-drained (CD, S) tests (either direct shear or triaxial) or consolidated-undrained (CU, R) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the "R" or "total stress" envelope and associated c and ö, from CU, R tests should not be used.

[b] For saturated soils use ö = 0; total stress envelope with ö > 0 is only applicable to partially saturated soils.

6-6. Minimum Acceptable Factors of Safety

The minimum required safety factors for the preceding design conditions along with the portion of the embankment for which analyses are required and applicable shear test data are shown in Table 6-1b.

6-7. Measures to Increase Stability

Means for improving weak and compressible foundations to enable stable embankments to be constructed thereon are discussed in Chapter 7. Methods of improving embankment stability by changes in embankment section are presented in the following paragraphs.

a. *Flatten embankment slopes.* Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation type failure that takes place entirely within the embankment. Flattening embankment slopes reduces gravity forces tending to cause failure, and increases the length of potential failure surfaces (and therefore increases resistance to sliding).

Table 6-1b
Minimum Factors of Safety - Levee Slope Stability

Type of Slope	Applicable Stability Conditions and Required Factors of Safety End-of-Construction	Long-Term (Steady Seepage)	Rapid Drawdown[a]	Earthquake[b]
New Levees	1.3	1.4	1.0 to 1.2	(see below)
Existing Levees	--	1.4[c]	1.0 to 1.2	(see below)
Other Embankments and dikes[d]	1.3[e,f]	1.4[c,f]	1.0 to 1.2[f]	(see below)

[a] Sudden drawdown analyses. F. S. = 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods preceding drawdown. F. S. = 1.2 applies to pool level, likely to persist for long periods prior to drawdown.

[b] See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.

[c] For existing slopes where either sliding or large deformation have occurred previously and back analyses have been performed to establish design shear strengths lower factors of safety may be used. In such cases probabilistic analyses may be useful in supporting the use of lower factors of safety for design.

[d] Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, river banks, and excavation slopes.

[e] Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. In such cases higher factors of safety may be required for end-of-construction to ensure stability during the time the excavation is to remain open. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.

[f] Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

b. Stability berms. Berms essentially provide the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing a substantial increase in the failure path. Thus, berms can be an effective means of stabilization not only for shallow foundation and embankment type failures but for more deep-seated foundation failures as well. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the embankment and prevent further movement.

6-8. Surface Slides

Experience indicates that shallow slides may occur in levee slopes after heavy rainfall. Failure generally occurs in very plastic clay slopes. They are probably the result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. These failures require maintenance and could be eliminated or reduced in frequency by using less plastic soils near the surface of the slopes or by chemical stabilization of the surface soils.

Section III
Settlement

6-9. General

Evaluation of the amount of postconstruction settlement that can occur from consolidation of both embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment. Many districts overbuild a levee by a given percent of its height to take into account anticipated settlement both of the foundation and within the levee fill itself. Common allowances are 0 to 5 percent for compacted fill, 5 to 10 percent for semicompacted fill, 15 percent for uncompacted fill, and 5 to 10 percent for hydraulic fill. Overbuilding does however increase the severity of stability problems and may be impracticable or undesirable for some foundations.

6-10. Settlement Analyses

Settlement estimates can be made by theoretical analysis as set forth in EM 1110-1-1904. Detailed settlement analyses should be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils. Where foundation and embankment soils are pervious or semipervious, most of the settlement will occur during construction. For impervious soils it is usually conservatively assumed that all the calculated settlement of a levee built by a normal sequence of construction operations will occur after construction. Where analyses indicate that more foundation settlement would occur than can be tolerated, partial or complete removal of compressible foundation material may be necessary from both stability and settlement viewpoints. When the depth of excavation required to accomplish this is too great for economical construction, other methods of control such as stage construction or vertical sand drains may have to be employed, although they seldom are justified for this purpose.

Design Criteria
Geotechnical Evaluation for Lower Silver Creek

TO:	George Fowler	
COPIES:	Design Consultants Jim Wang/SCVWD Mohammed Khan/SCVWD Dick Volpe/SCVWD	Bill Ward/NRCS Steve Wrightson/CWP John Anderson/CWP Jay Aldean/CWP
FROM:	Constance Eichhorn/CWP	
DATE:	April 3, 2001	

Objective

To provide stable slopes (preferably 2:1 natural slopes) for a 100-year storm event that require little or no rehabilitation from sloughing during a major seismic event.

Purpose

This memo provides direction on the following design issues:

- Explanation for a deformation analysis (see attached memo by John Anderson)
- Guidelines to use in assessing allowable deformation

Background

The District has developed a design criteria for analyzing slopes of the channel for dam and levee conditions (where there is constant hydrostatic pressures in the slope—i.e constant phreatic surface for retained water). The design criteria includes a seismic component. Utilizing this standard seismic criteria for an incised channel with little or no hydrostatic pressures in the slope is considered to be overly conservative.

A Value Engineering (VE) analysis was performed in October 2000 for Lower Silver Creek Project Reaches 1-2. Utilizing the existing District geotechnical criteria would result in 2.5:1 or 3:1 side slopes for natural earthen channel. It was estimated that this criteria alone increased the cost of Reach 1 project by nearly 20 percent because of the additional excavation required. Using flatter slopes also resulted in a decrease in upland planting area and more encroachment onto private property.

The existing natural channel appears to be relatively stable and is standing at slopes of 1:1 or steeper. The VE recommendation was to develop a new seismic design criteria that would be more applicable for an incised channel that may only be subject to full hydrostatic loading for short durations and infrequently (8 hours during a 100 year storm event).

Design Criteria

A discussion of the deformation analysis is provided in Attachment A.

There is agreement that some deformation should be allowed for the unlikely event that a seismic event occurs during a 100-year storm event. The risk of these two events occurring simultaneously is low. Even if this set of circumstances should occur, the resulting deformation would not result in a catastrophic failure since there is no water being retained behind a dam or levee that would cause flooding. At worse the slope may slough off and have to be repaired in localized areas.

The recommended criteria for allowable deformation is as follows:

1) Design consultant shall perform a stability analysis (deformation analysis) for the existing channel configuration using surveyed cross sections of the creek at critical locations.

2) Design consultant shall perform deformation analysis for the proposed project improvements (preferably 2:1 natural slopes) including all other District provided direction (eliminate low flow channel and create tiered slope per design criteria memo dated March 26, 2000).

3) Consultant shall evaluate the "before" and "after project" condition. Under NO circumstances shall the deformation be greater than the "before" or existing condition.

4) The allowable displacement for the "after project" condition shall be limited to the following:

    a) 1.0 foot maximum horizontal displacement in open undeveloped areas (i.e. parks District owned lands etc.)

    b) 0.25-0.5 inches maximum horizontal displacement where there are structures located within 20 feet of the new top of bank

    c) 0.25-0.5 inches where there is a likelihood that future structures may be constructed on private property within 20 feet of the channel (i.e. future residential development). The alternative to this approach would be to place restrictions on the development/construction of any structures within 20 feet of the proposed top of bank regardless of the property boundary through the permitting process.

5) The Consultant is directed to coordinate with District staff where there are structures within 20 feet of the proposed top of bank to determine acceptable deformations and risk to the District.

Based on the direction provided above, the resulting slopes may vary through the project to minimize the risk of damage to structures on private property.

Attachment A
Slope Stability Design Criteria

The Coyote Creek Watershed Program involves flood protection improvements of existing stream channels within the watershed. The improvements generally require the modification of levees and natural slopes along the channels.

As part of the design of improvements the consultants' for the project are performing slope stability evaluations of the modifications to the natural slopes and existing levees. Where this is necessary the criteria presented in Table 1 should be used.

The criteria is based on the US Army Corps of Engineers Engineer Manual EM 1110-2-1913, Design and Construction of Levees (April 30, 2000). The criteria has been modified to reflect the use of a horizontal (pseudo static) lateral coefficient of 0.2g for Earthquake evaluation.

Also, for "Other Slopes" (natural slopes and cut slopes) the criteria for Earthquake evaluation has been modified to allow a deformation analysis in cases where the pseudo static analysis indicates a Factor of Safety less than one. If a deformation analysis is performed it should estimate the displacement associated with a peak acceleration of 0.6 g (which represents the peak acceleration associated with the maximum credible earthquake for the project area (California Seismic Hazard Map 1996, Office of Earthquake Engineering, California Department of Transportation)). The impact of the estimated deformation should then be determined based on a review of adjacent facilities.

The movement estimated using the deformation approaches is assumed to occur tangent to the slip surface, and thus the estimated displacement has both a horizontal and vertical component. Generally, displacements on the order of 1 foot can be tolerated when the potential sliding mass is in an area where there is no development (or anticipated development). If structures or utilities are present the impact of the estimated displacement needs to be assessed. Occupied buildings and stiff improvements such as patios, pools, etc. may not generally tolerate movements greater than 0.25-0.5 inches. In order to assist in this evaluation, the movement that would occur to the design slope configuration should be compared to the movement that is estimated for the existing slope configuration.

Although the simplified deformation approach was initially developed to estimate the displacement of dams, it has been used to evaluate displacements for both natural and cut slopes. It is used for highway, port and pipeline projects where seismic deformations of slopes may induce movements which can impact the constructed facilities. The period of the slope can be estimated by determining the thickness of the failure wedge or slip zone and computing the period from the relationship: $T = 4H/Vs$. Where T is the period; H is the thickness of failure zone; and Vs is the estimated shear wave velocity of the soil.

Table 1 Design Criteria -- Slope Stability Minimum Factors of Safety

	Applicable Stability Conditions and Required Factors of Safety			
Type of Slope	End of Construction	Long Term (Steady Seepage)	Rapid Drawdown[1]	Earthquake[2]
New Levees	1.3	1.4	1.0 to 1.2	1.0
Existing Levees	---	1.4[3]	1.0 to 1.2	1.0
Other Slopes[4]	1.3	1.4[3]	1.0 to 1.2	1.0[5]

---

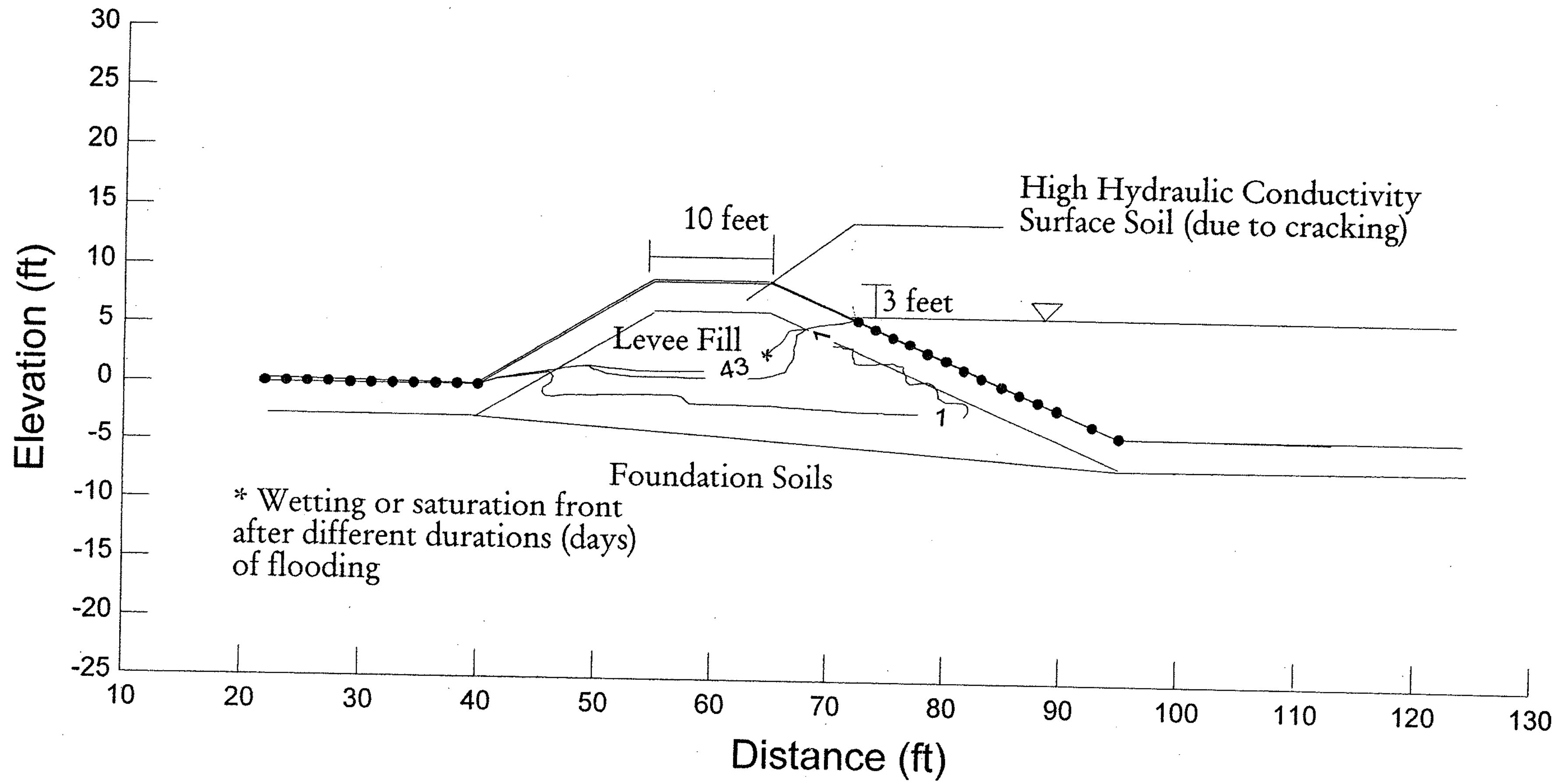
[1] Rapid drawdown analyses. F.S. = 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods (sufficient time for a phreatic surface consistent with pool level to develop within the slope--i.e. storm surges) preceding drawdown. F.S. = 1.2 applies to pool level, likely to exist for long periods (water is maintained so that a phreatic surface consistent with the pool level develops within the slope--steady state seepage) prior to drawdown.

[2] For seismic loading assume a pseudo static lateral coefficient of 0.2 g

[3] For existing slopes where either sliding or large deformation have previously occurred and back analyses have been performed to establish design shear strengths, lower factors of safety may be used.

[4] Includes slopes which are natural bank slopes of creeks or rivers and cut slopes.

[5] Determine the yield acceleration (pseudo static lateral coefficient which results in a factor of safety equal to 1.0) for the slope. Start with an pseudo static coefficient of 0.15 g and increase in increments of 0.05g until yield acceleration is determined. Perform a deformation analysis to estimate the displacement associated with the maximum probable earthquake event. If facilities impacted can tolerate the estimated movements, then a factor of safety less than 1.0 is acceptable.
If the potential sliding mass is within District Right of Way (ROW) and no existing facilities are impacted, then estimated displacements on the order of one foot can be accepted for the design. If the potential sliding mass extends outside of District ROW, then the estimated displacements of the design should not exceed those estimated for the existing slope configuration.



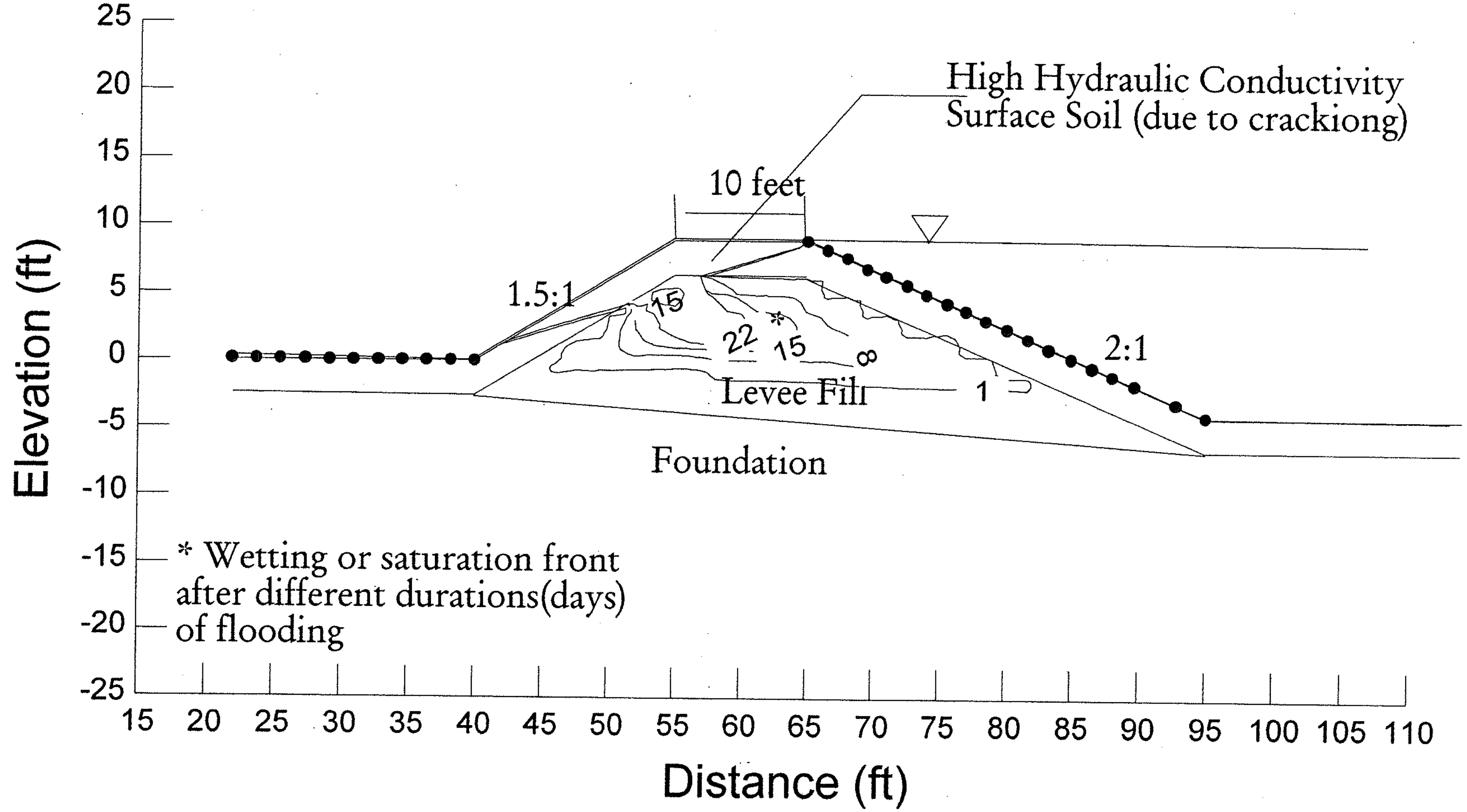
6/01*EB

SEEPAGE ANALYSIS-AVERAGE EXISTING CONDITIONS, FLOOD WITH FREEBOARD

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX E-1
1697-1



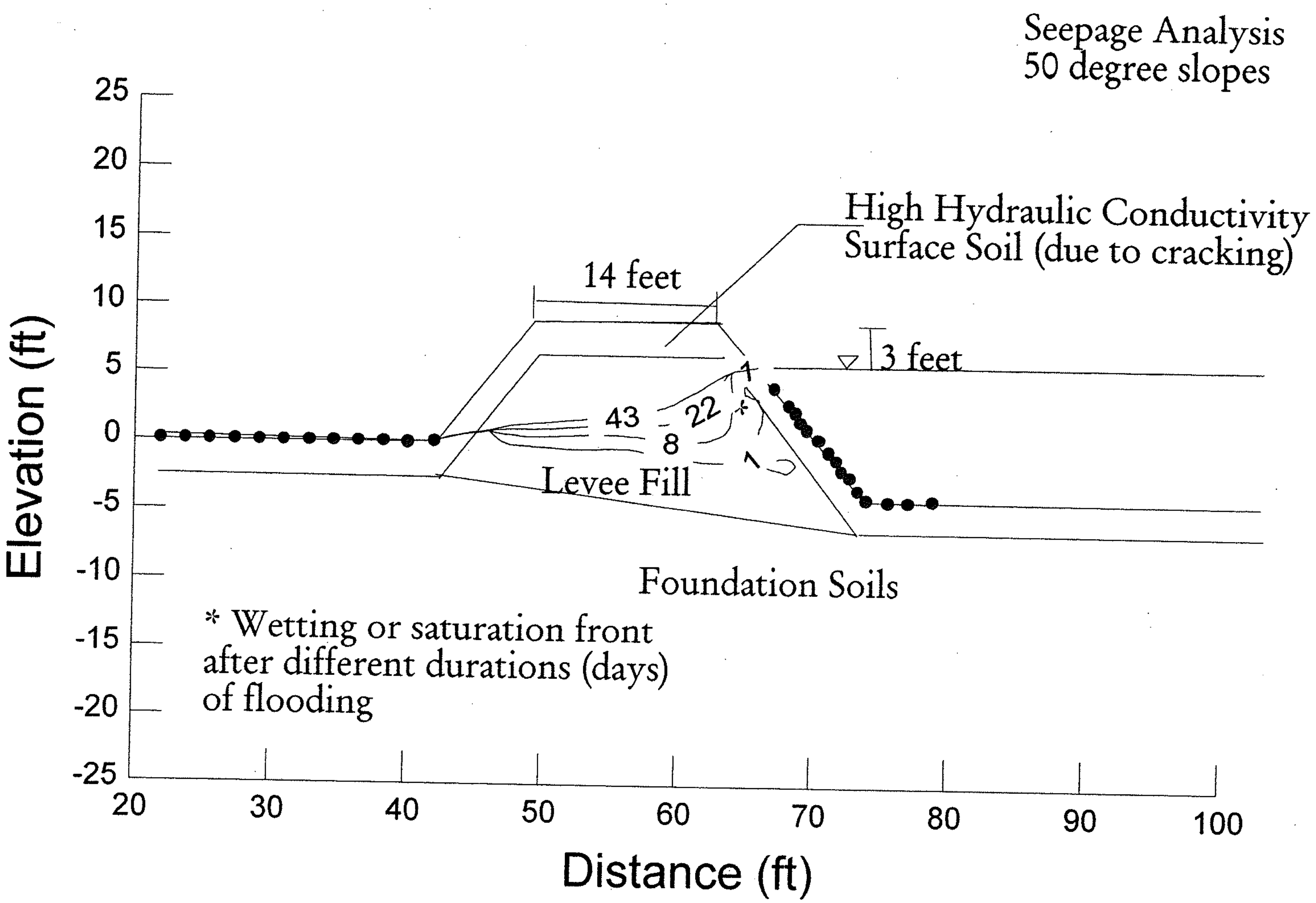
6/01*EB

SEEPAGE ANALYSIS-AVERAGE EXISTING CONDITIONS, FLOOD NO FREEBOARD

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX E-2
1697-1



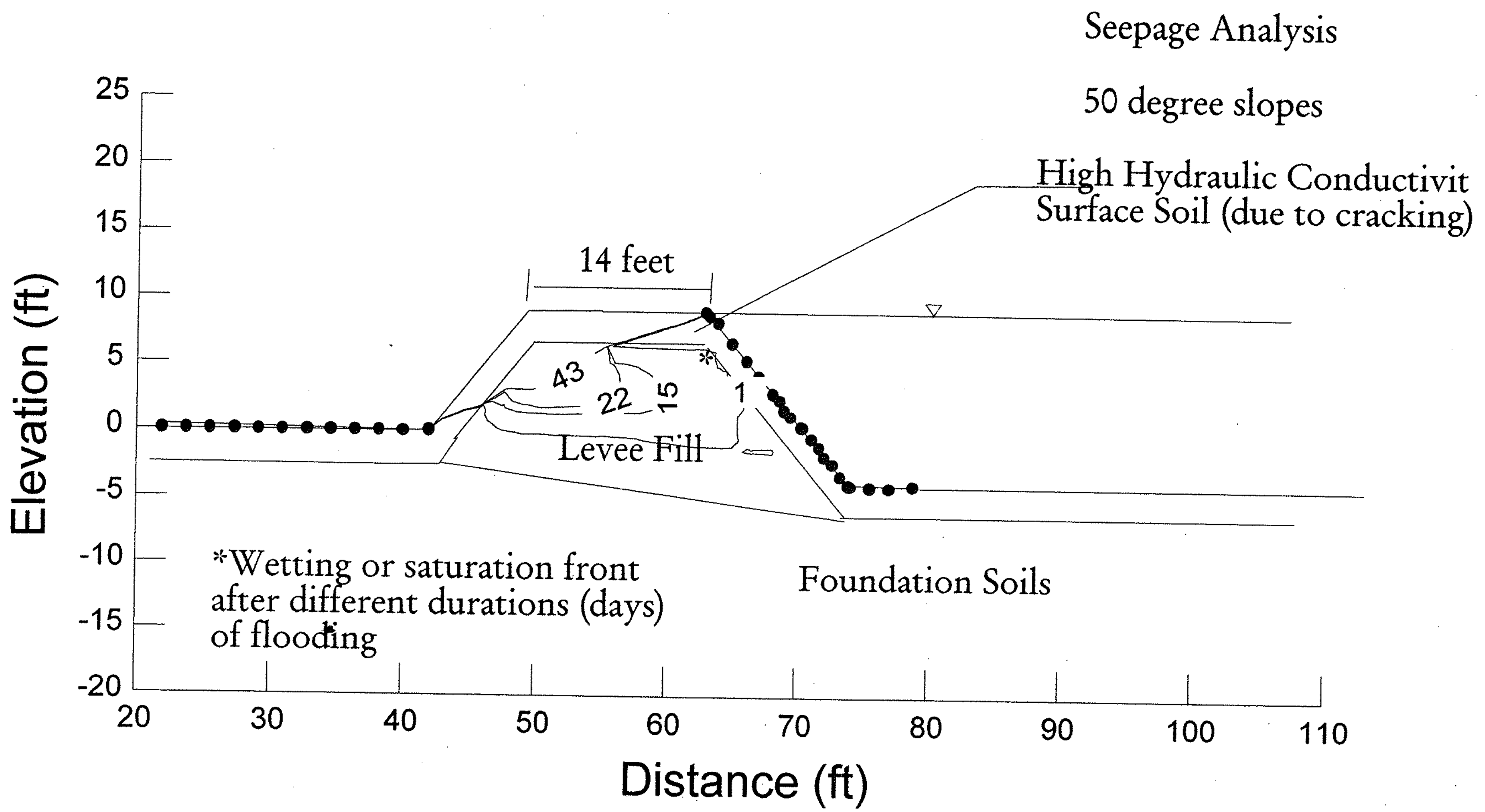
6/01*EB

SEEPAGE ANALYSIS-50 DEGREE SLOPES, FLOOD WITH FREEBOARD

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX E-3
1697-1



6/01*EB

SEEPAGE ANALYSIS-50 DEGREE SLOPES, FLOOD NO FREEBOARD

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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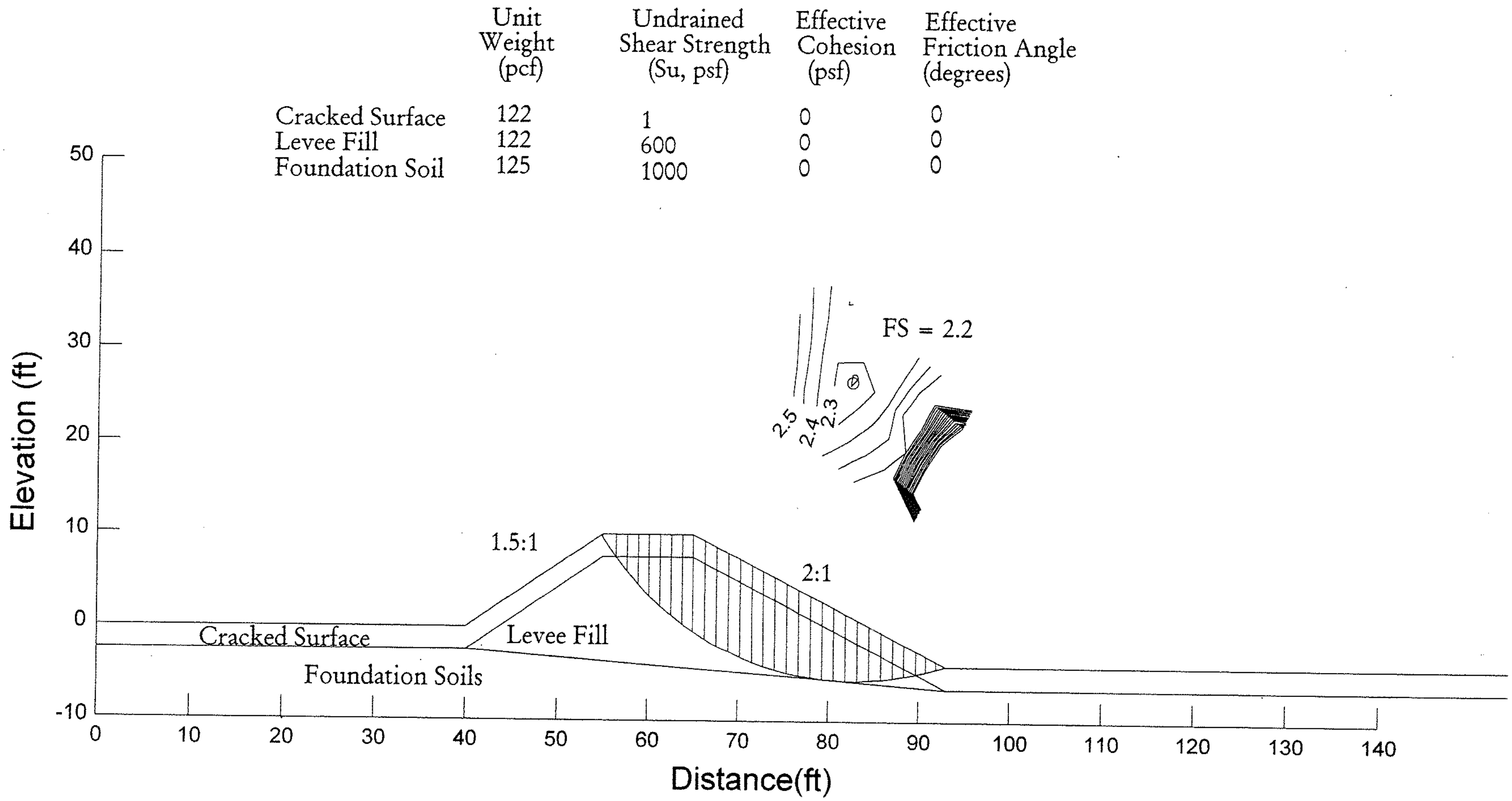
Environmental/Geotechnical/Engineering Services

APPENDIX E-4
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0

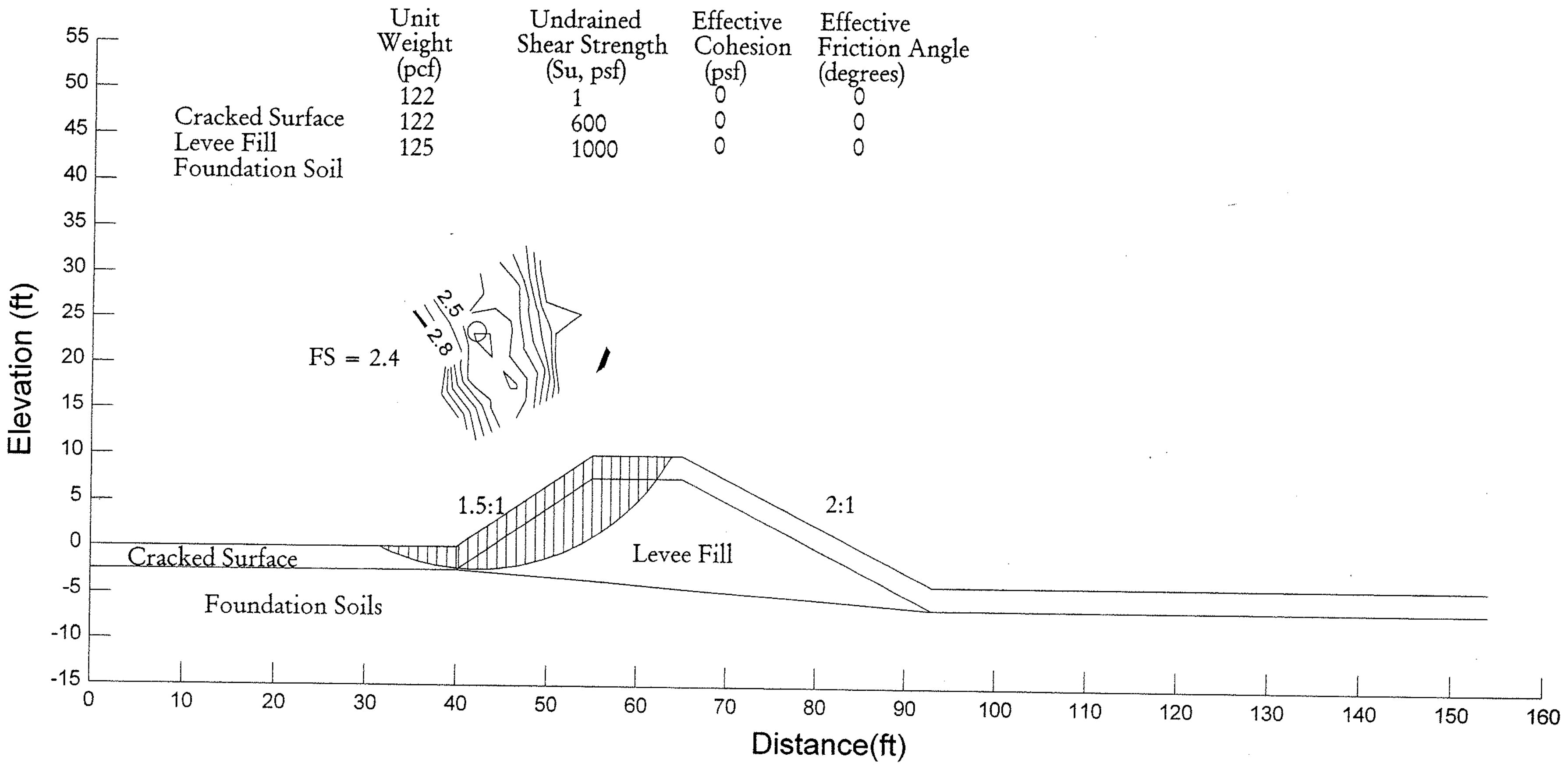
8/01*EB

SHORT TERM WITHOUT FLOODING (CASE I), 2:1 CREEKSIDE SLOPES, FOUNDATION s_u = 1000

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-1
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0

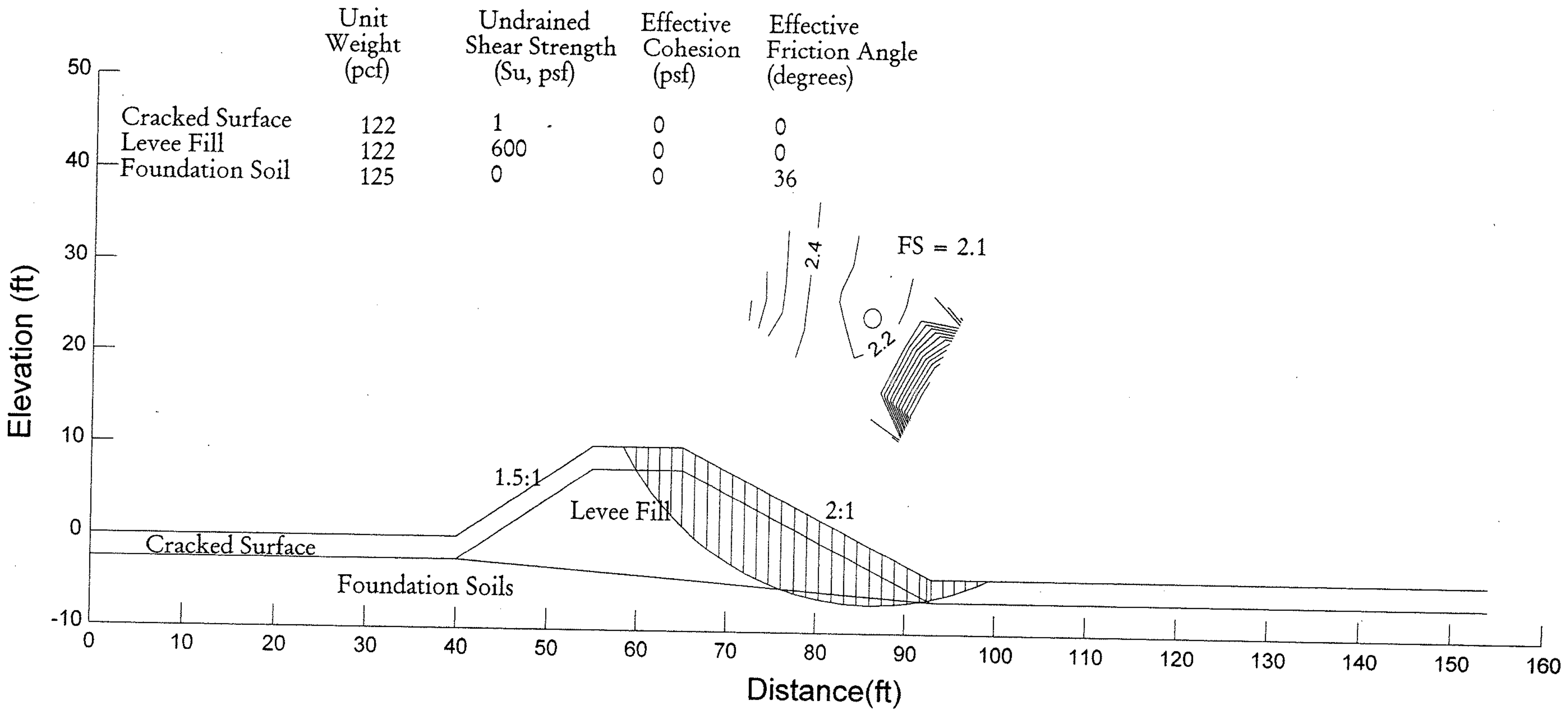
8/01*EB

SHORT TERM WITHOUT FLOODING (CASE I), 1.5:1 DRYSIDE SLOPES, FOUNDATION s_u = 1000

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-2
1697-1



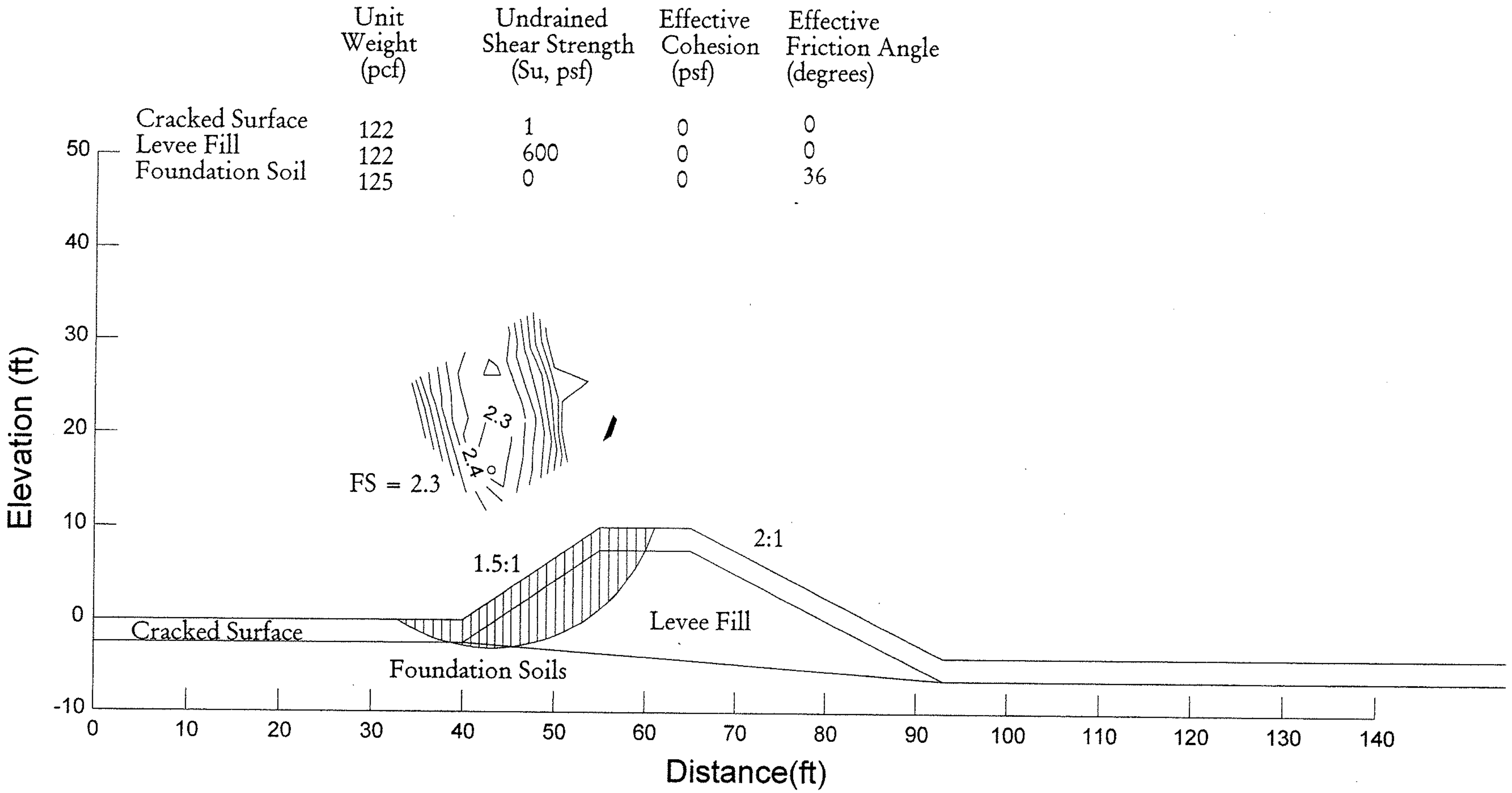
6/01*EB

LONG TERM WITHOUT FLOODING, 2:1 CREEKSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-3
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	0	0	36

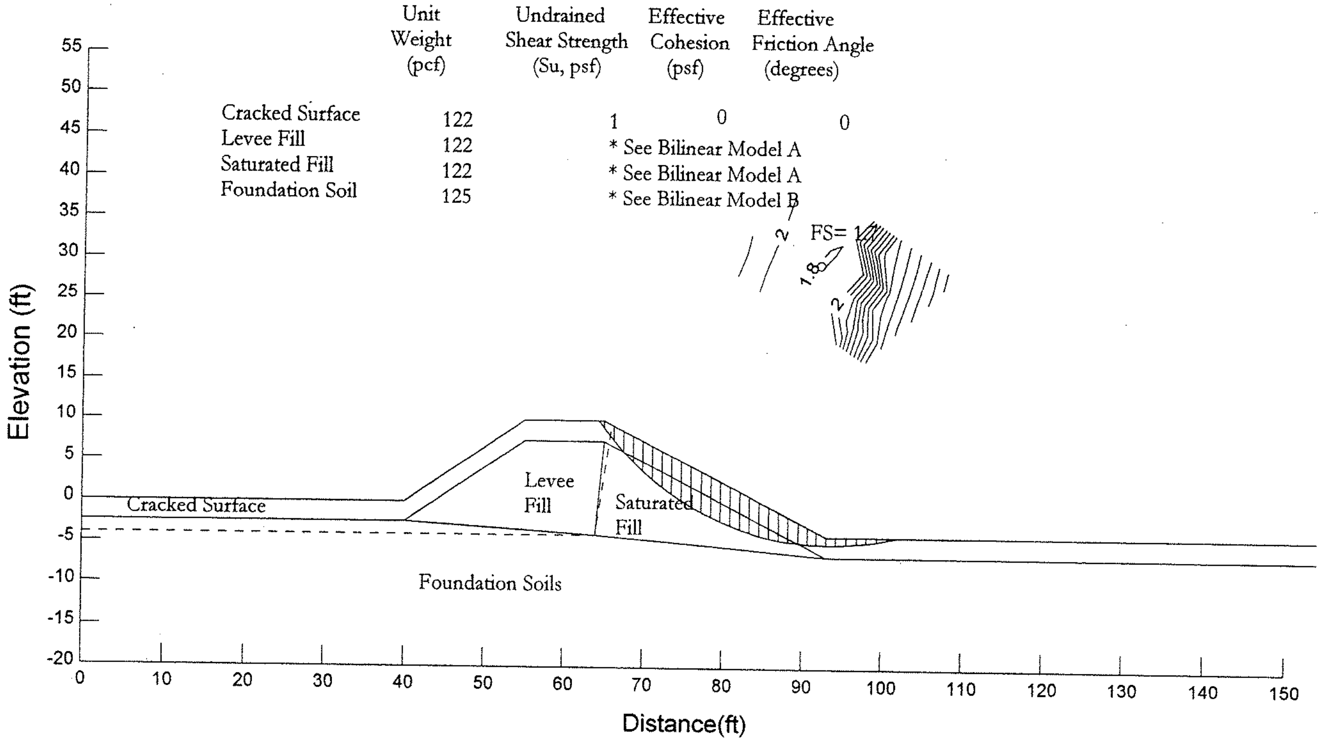
6/01*EB

LONG TERM WITHOUT FLOODING, 1.5:1 CREEKSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

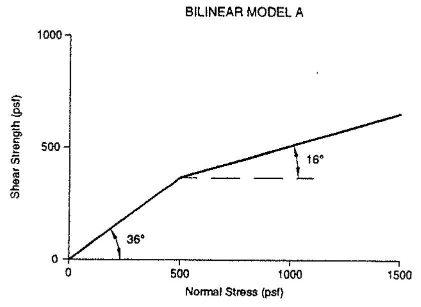
BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

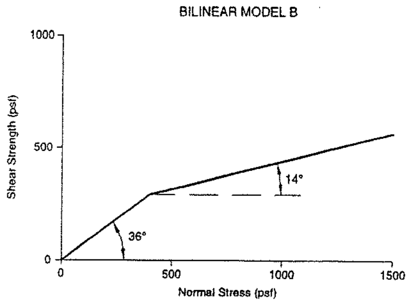
LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-4
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	* See Bilinear Model A		
Saturated Fill	122	* See Bilinear Model A		
Foundation Soil	125	* See Bilinear Model B		





8/01*EB

SUDDEN DRAWDOWN (CASE II), CREEKSIDE SLOPE

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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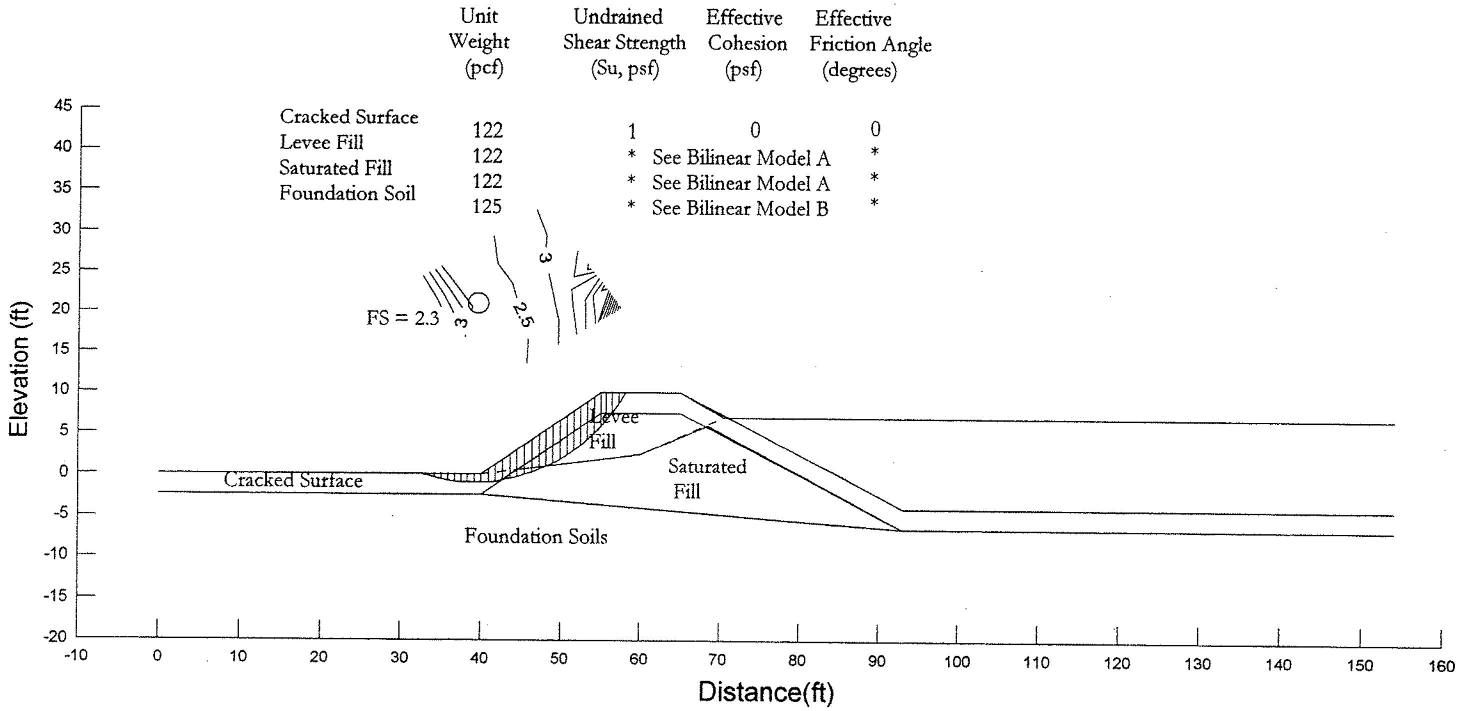
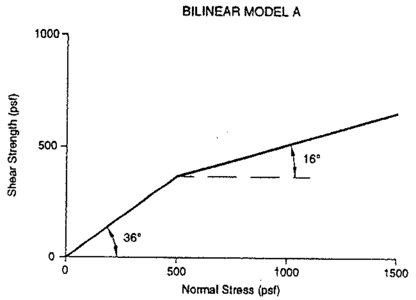
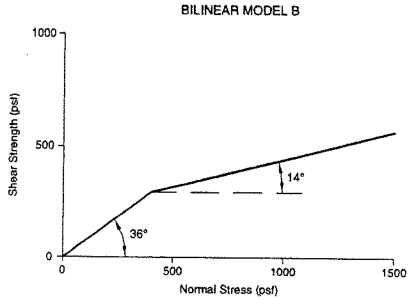
Environmental/Geotechnical/Engineering Services

APPENDIX F-5
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	*	See Bilinear Model A	*
Saturated Fill	122	*	See Bilinear Model A	*
Foundation Soil	125	*	See Bilinear Model B	*





8/01*EB

STEADY SEEPAGE (CASE III), FROM FULL FLOOD

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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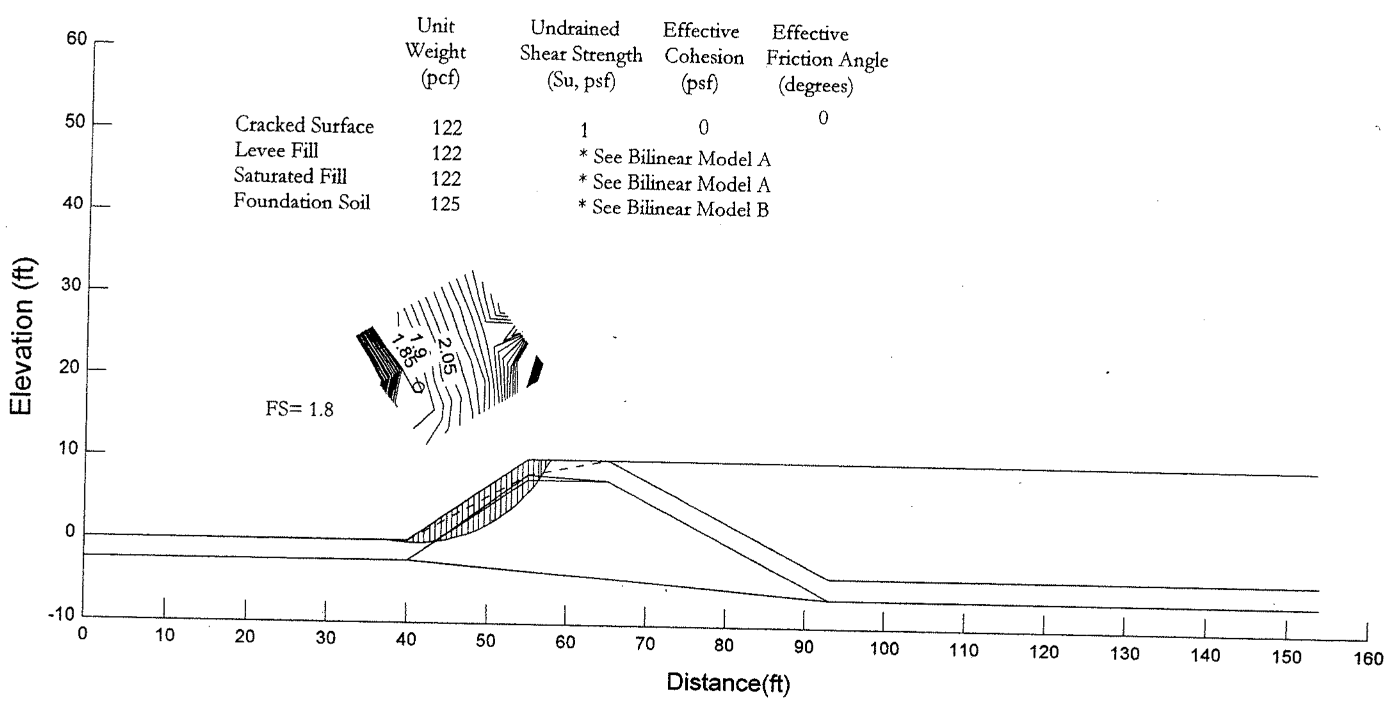
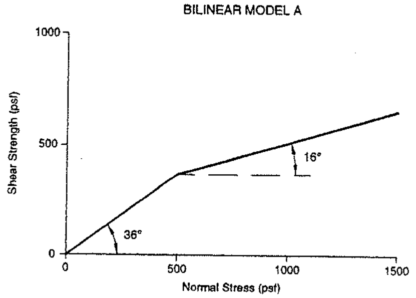
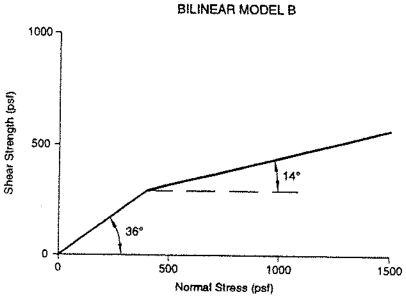
Environmental/Geotechnical/Engineering Services

APPENDIX F-6
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	* See Bilinear Model A		
Saturated Fill	122	* See Bilinear Model A		
Foundation Soil	125	* See Bilinear Model B		





8/01*EB

STEADY SEEPAGE (CASE III), FLOOD AT TOP OF LEVEE

BERRYESSA CREEK LEVEES PROJECT

Milpitas, California

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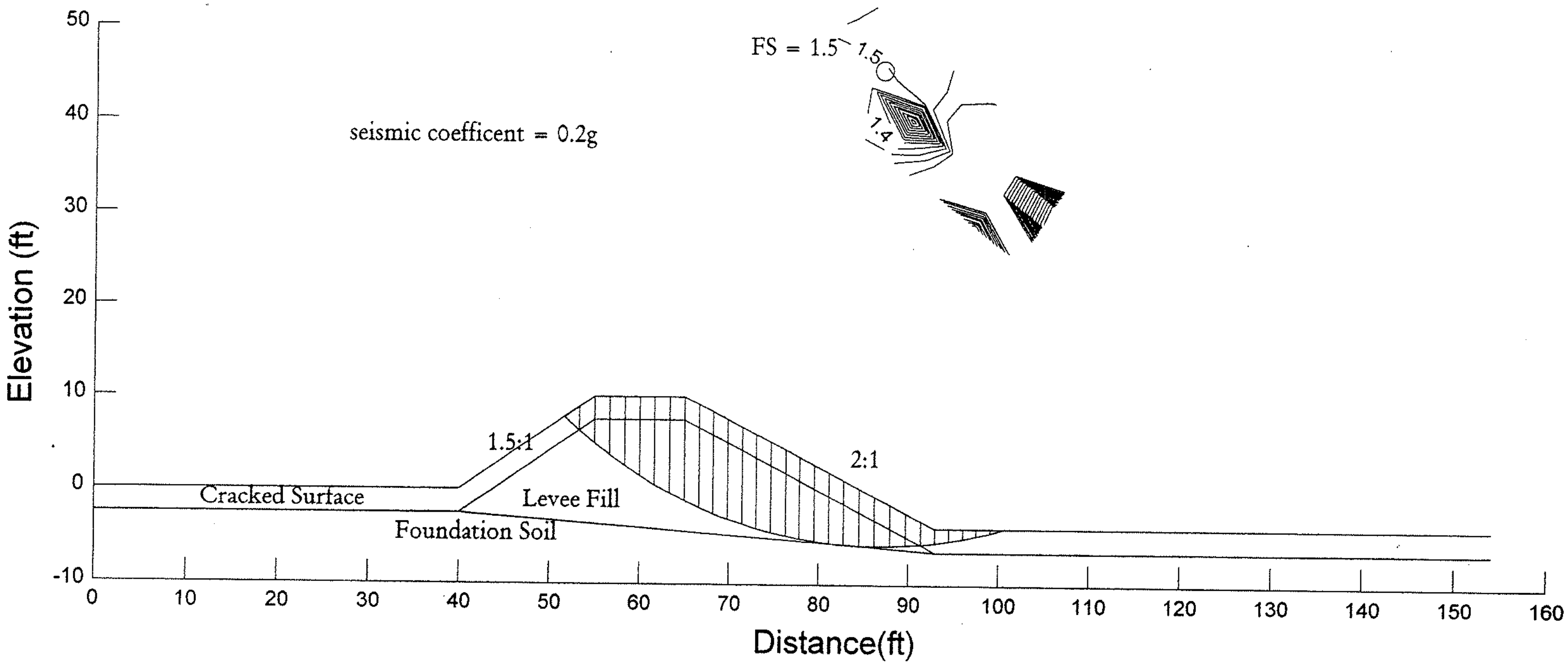
Environmental/Geotechnical/Engineering Services

APPENDIX F-7
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0



8/01*EB

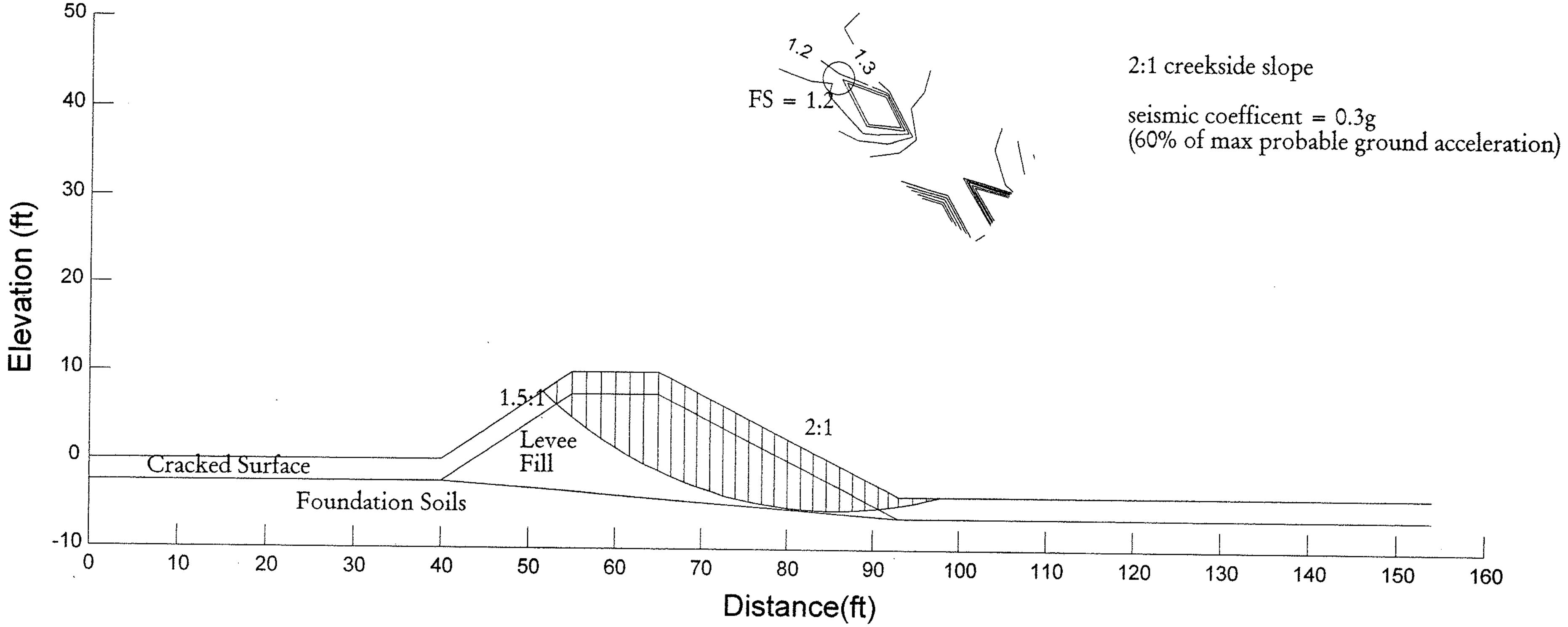
SHORT TERM WITH EARTHQUAKE (CASE IV), 2:1 CREEKSIDE SLOPES, FOUNDATION s_u = 1000

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-8
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0



2:1 creekside slope

seismic coefficent = 0.3g
(60% of max probable ground acceleration)

8/01*EB

SHORT TERM WITH EARTHQUAKE (CASE IV), 2:1 CREEKSIDE SLOPES, FOUNDATION s_u = 1000

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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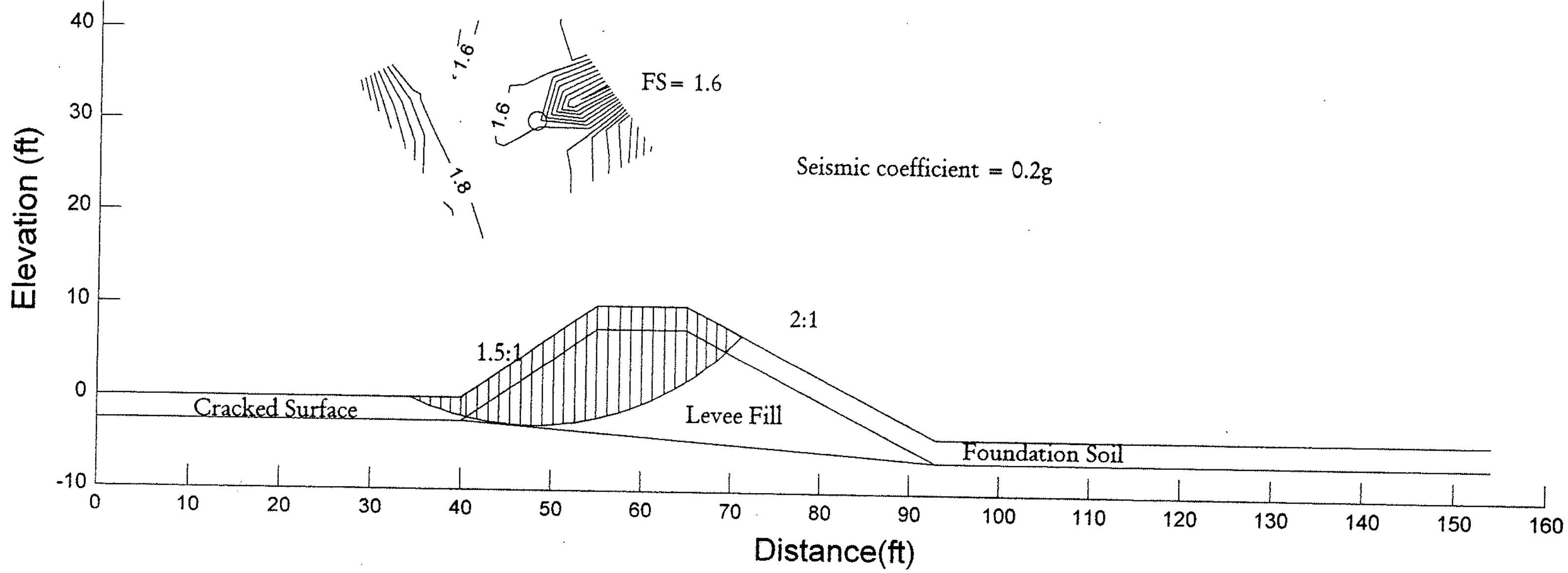
Environmental/Geotechnical/Engineering Services

APPENDIX F-9
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0



8/01*EB

SHORT TERM WITH EARTHQUAKE (CASE IV), 1.5:1 DRYSIDE SLOPES, FOUNDATION $s_u = 1000$

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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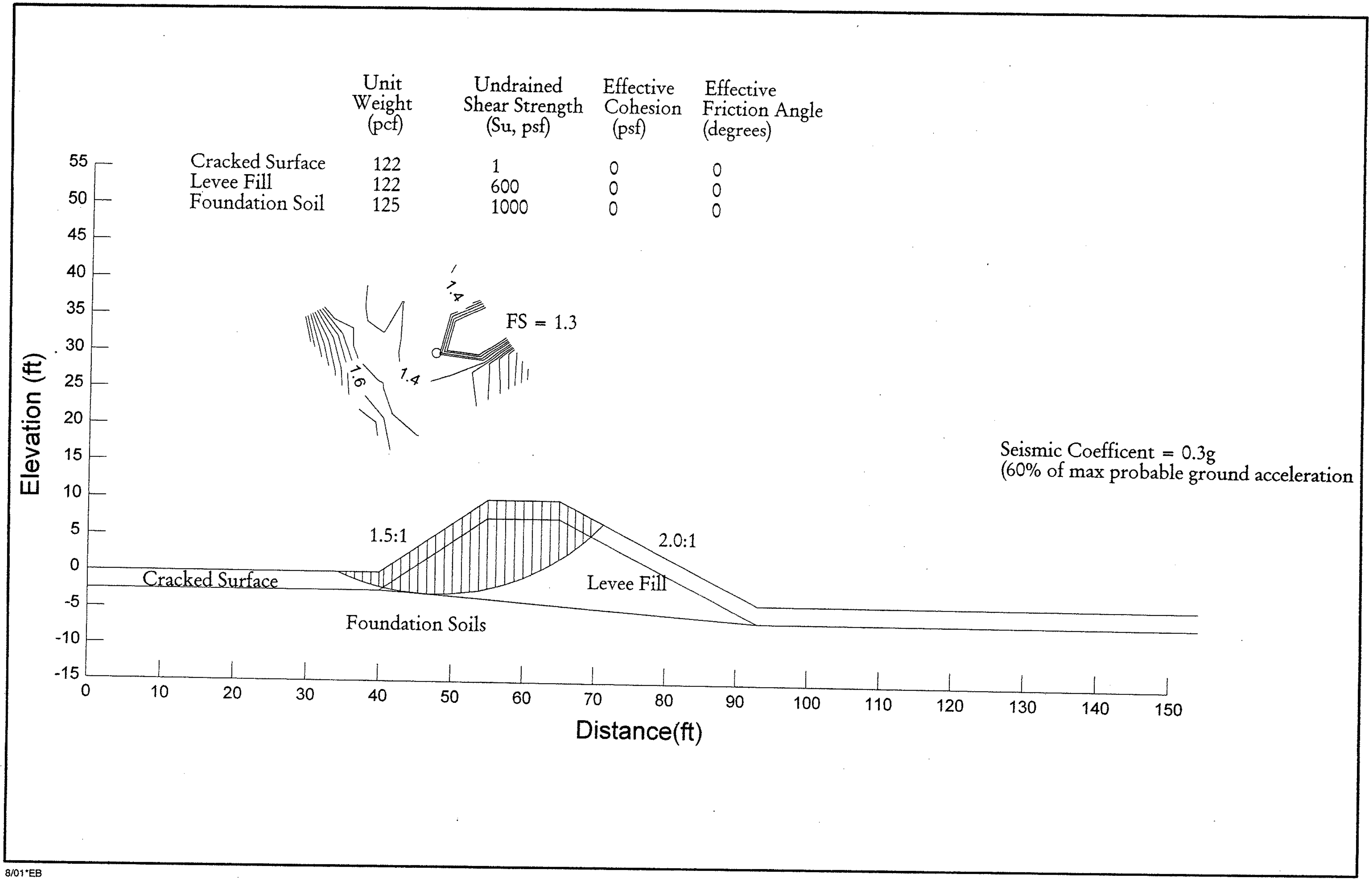
Environmental/Geotechnical/Engineering Services

APPENDIX F-10
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	1000	0	0

8/01*EB

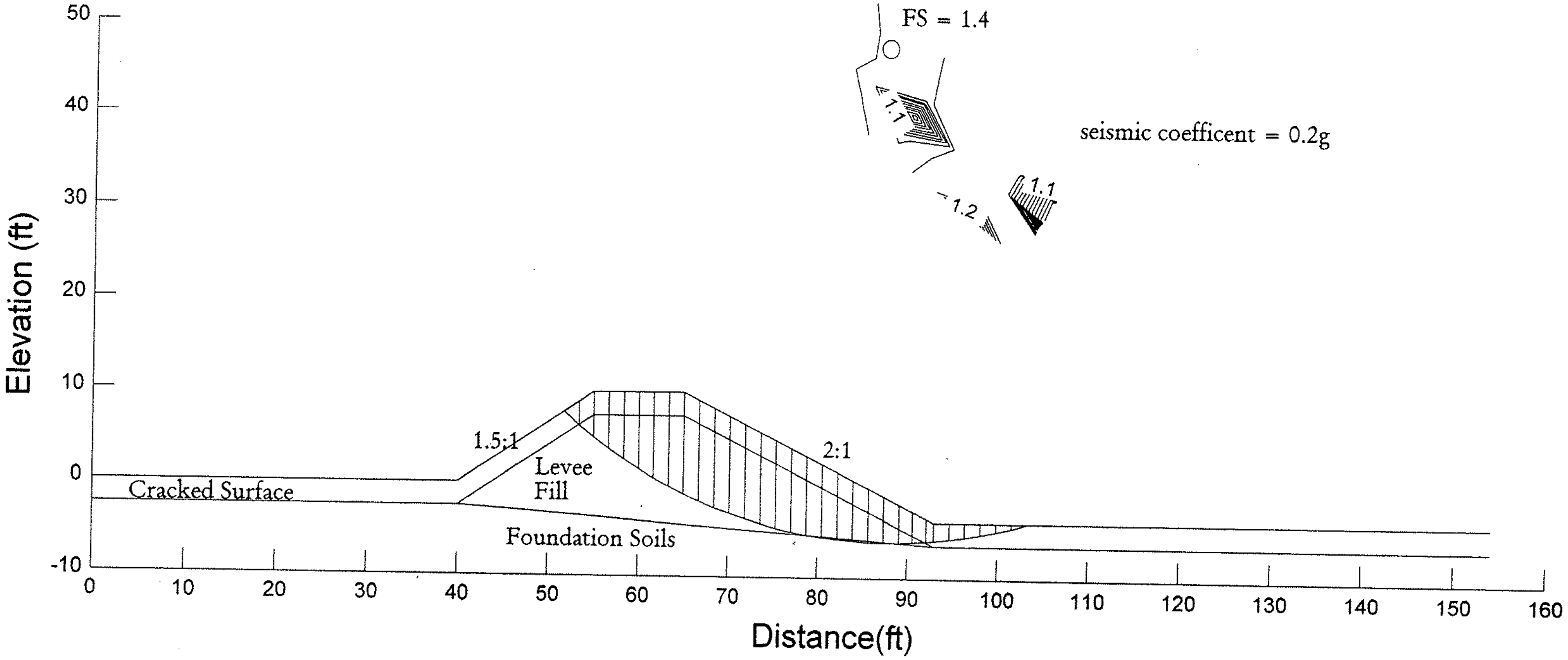
SHORT TERM WITH EARTHQUAKE (CASE IV), 1.5:1 DRYSIDE SLOPES, FOUNDATION s_u = 1000

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-11
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	0	0	36



6/01*EB

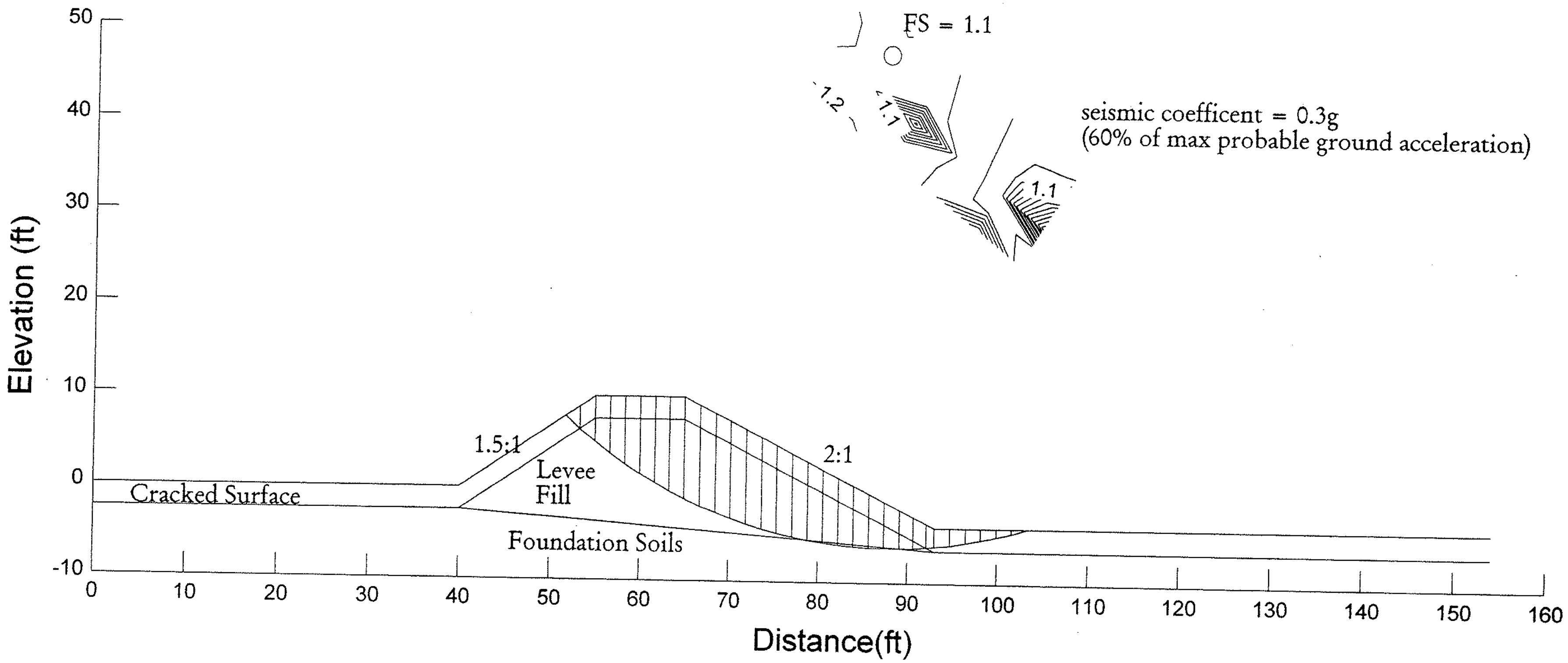
LONG TERM WITH EARTHQUAKE (CASE IV) , 2:1 CREEKSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-12
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	0	0	36



6/01*EB

LONG TERM WITH EARTHQUAKE (CASE IV) , 2:1 CREEKSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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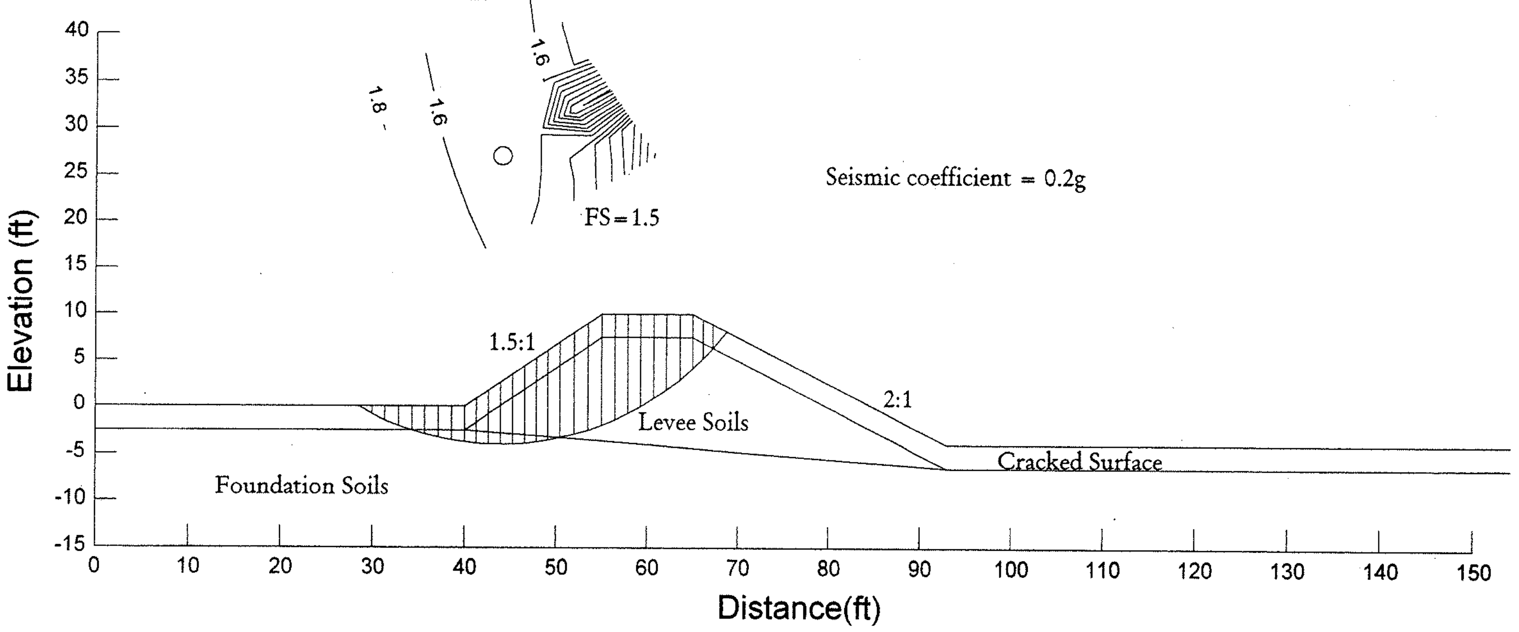
Environmental/Geotechnical/Engineering Services

APPENDIX F-13
1697-1

	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	0	0	36



6/01*EB

LONG TERM WITH EARTHQUAKE (CASE IV), 1.5:1 DRYSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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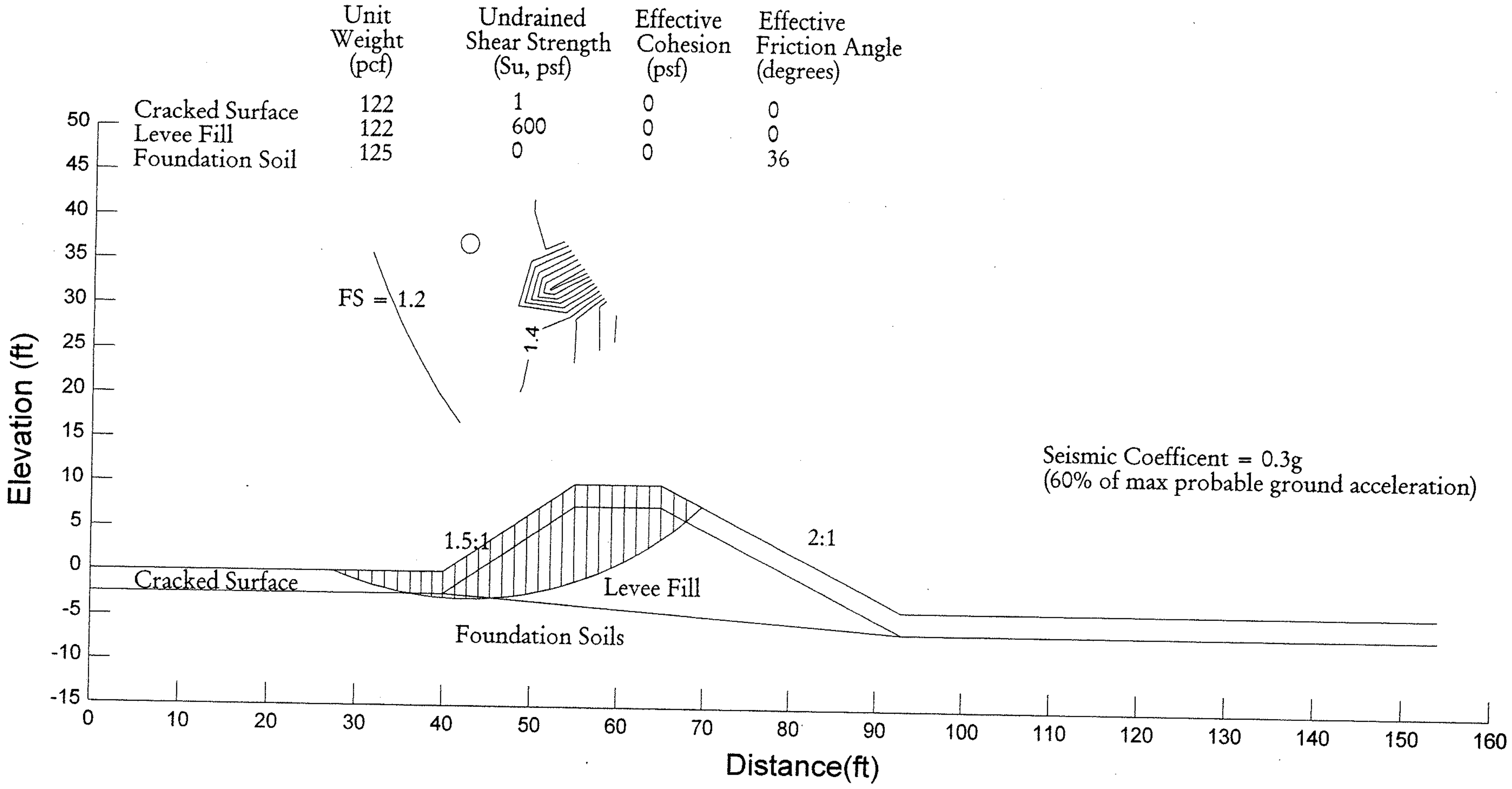
Environmental/Geotechnical/Engineering Services

APPENDIX F-14
1697-1



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Cracked Surface	122	1	0	0
Levee Fill	122	600	0	0
Foundation Soil	125	0	0	36

6/01*EB

LONG TERM WITH EARTHQUAKE (CASE IV), 1.5:1 DRYSIDE SLOPES, FOUNDATION EFFECTIVE ANGLE = 36°

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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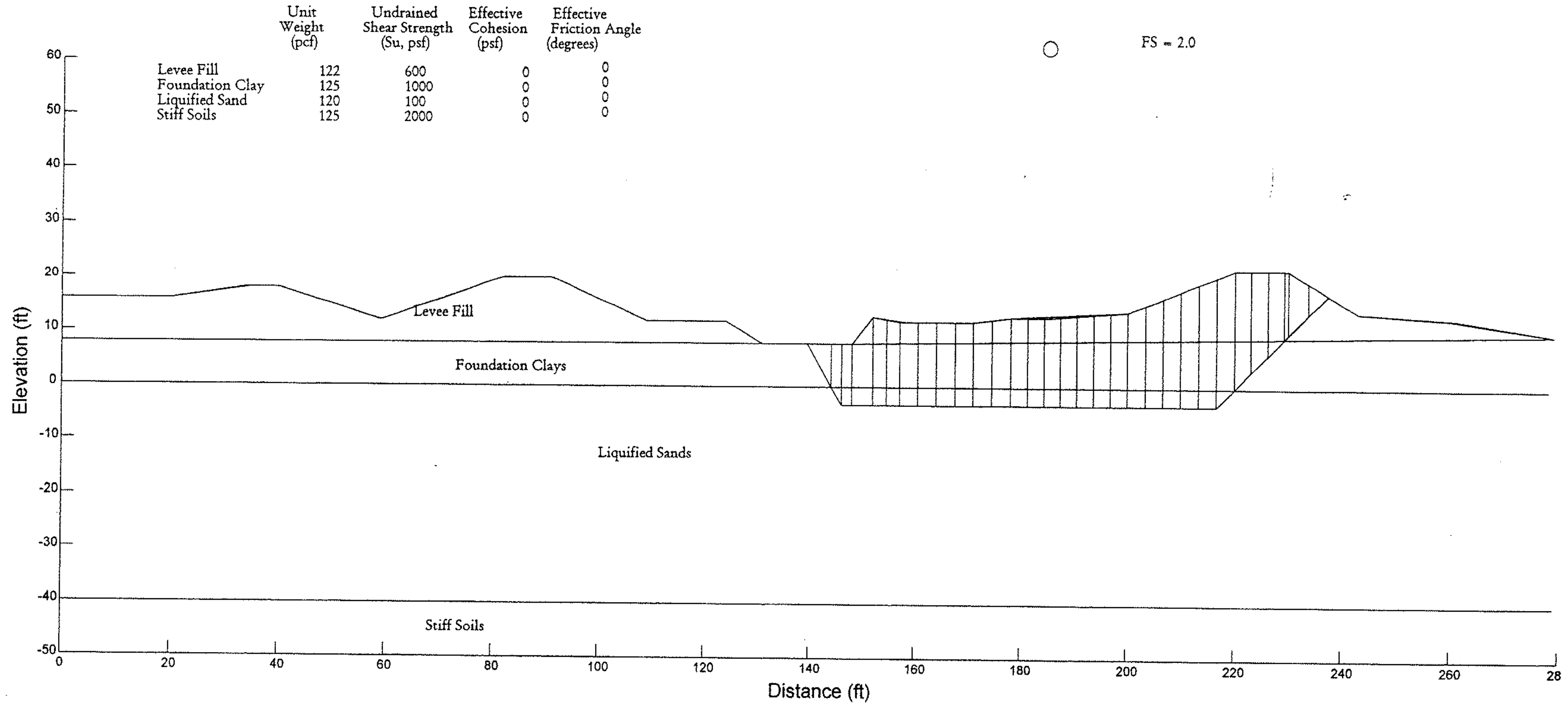
Environmental/Geotechnical/Engineering Services



	Unit Weight (pcf)	Undrained Shear Strength (Su, psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)
Levee Fill	122	600	0	0
Foundation Clay	125	1000	0	0
Liquified Sand	120	100	0	0
Stiff Soils	125	2000	0	0

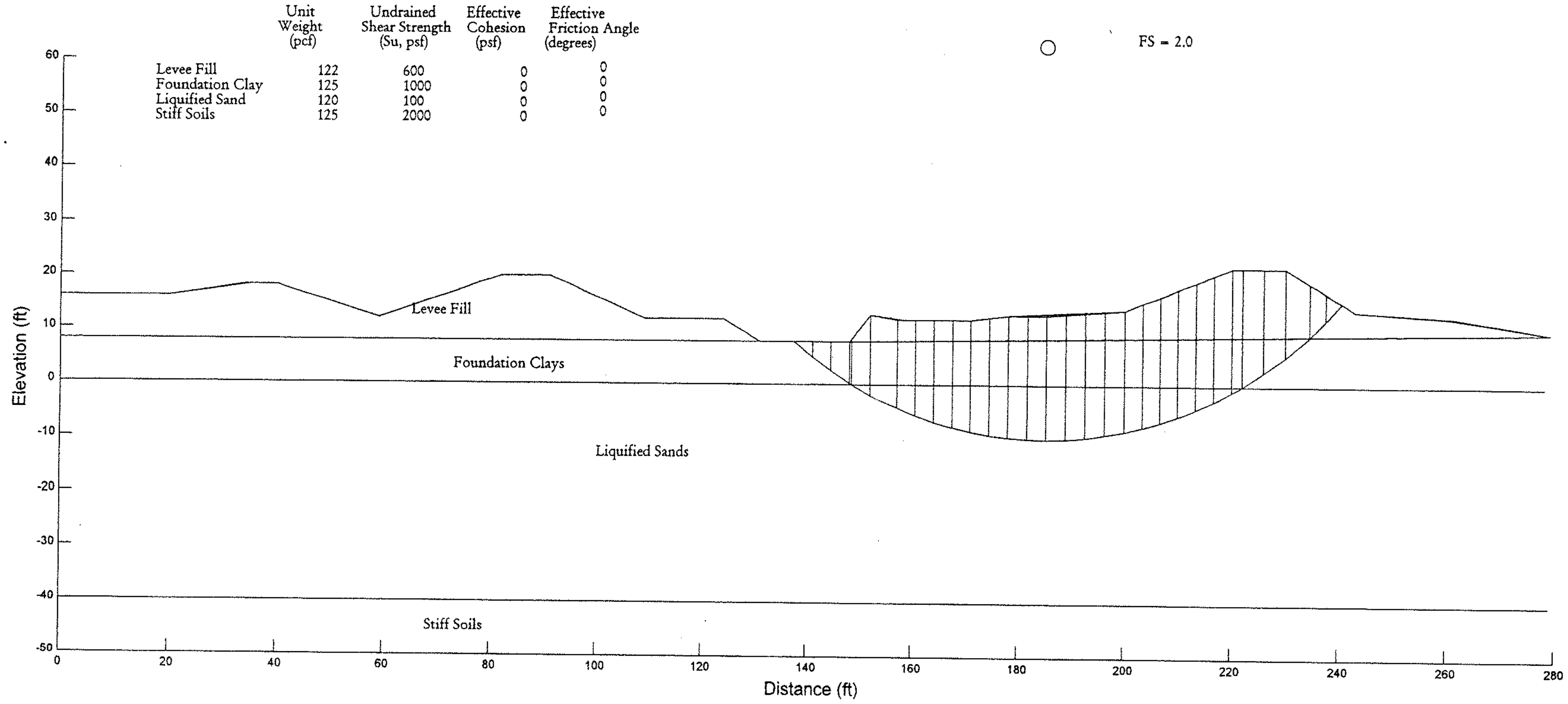
8/01*EB

LIQUEFACTION OF SAND LAYERS, NO EARTHQUAKE FORCE-BLOCK FAILURE

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-16
1697-1



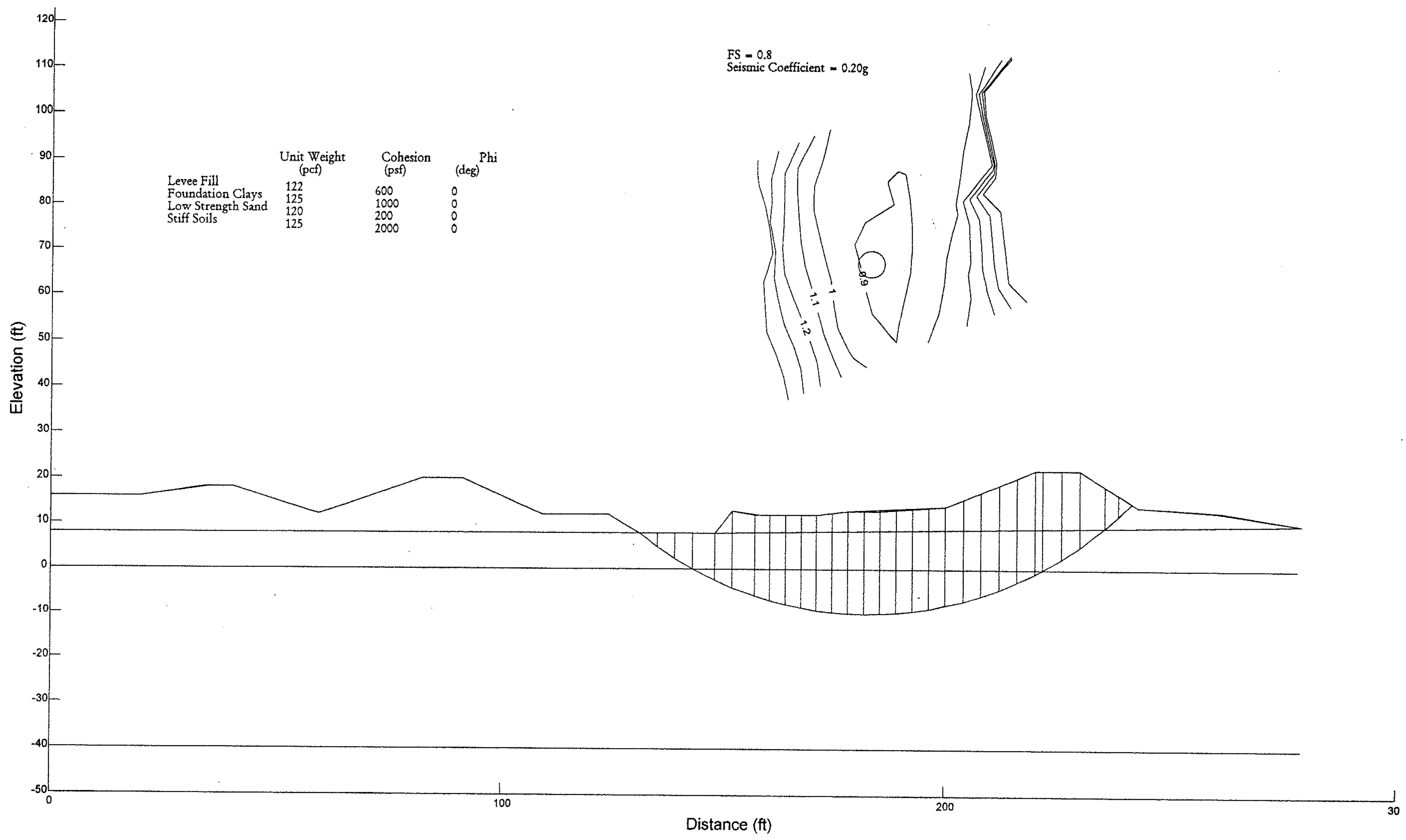
8/01*EB

LIQUEFACTION OF SAND LAYERS, NO EARTHQUAKE FORCE-CIRCLE FAILURE

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

APPENDIX F-17
1697-1



8/01*EB

LIQUEFACTION OF SAND LAYERS, WITH 0.2g EARTHQUAKE FORCE-CIRCLE FAILURE

BERRYESSA CREEK LEVEES PROJECT
Milpitas, California

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Environmental/Geotechnical/Engineering Services

APPENDIX F-18
1697-1

Gutierrez Copy
~~File~~

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

Memorandum

JAN - 4 2002

To: Steve Cox, Winzler & Kelly

From: Fred Brovold

Date: January 3, 2002

Re: Berryessa Creek Levees Project, Laboratory Test Data Supplement for Preliminary Geotechnical Report

---

Please find attached a draft copy of a supplement to the Preliminary Geotechnical Report. The supplement provides the additional laboratory test data that is discussed in Section 3.1.8 of the report.

Please contact us at 650 967-2365, x-122, or fbrovold@lowney.com if you have any questions or require further assistance.

Env/forms/1697-1_Memo01.doc

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

Mountain View
Oakland
San Ramon
Fullerton

January 3, 2002
1697-1

Mr. Steve Cox, P.E.
WINZLER & KELLY
465 Tesconi Drive
Santa Rosa, California 95401

RE: SUPPLEMENT NO.1
SOIL PROPERTY AND INTERFACE
DIRECT SHEAR TESTING
PRELIMINARY GEOTECHNICAL REPORT
BERRYESSA CREEK LEVEES PROJECT
SANTA CLARA VALLEY WATER DISTRICT
MILPITAS, CALIFORNIA

Dear Mr. Cox:

This supplement to our Preliminary Geotechnical Report dated August 28, 2001 provides the geogrid interaction coefficient test data discussed in Section 3.1.8 of the report. The testing was performed by SGI Testing Services, LLC, Atlanta, Georgia. If you have any questions regarding the test results, please contact us and we will be pleased to discuss them with you.

Sincerely,

LOWNEY ASSOCIATES

DRAFT

Fred N. Brovold, P.E., G.E.
Senior Project Engineer

FNB:fnb

Copies: Addressee (3)
Santa Clara Valley Water District (3)
Attn: Mr. Jose Guiterez

MV, P:\Projects\1600\1697-1\1697-1_GDR_Supp01.doc

JAN - 4 2002

405 Clyde Avenue Mountain View, CA 94043-2209 Tel: 650.967.2365 Fax: 650.967.2785
P.O. Box 1388 Mountain View, CA 94042-1388 E-mail: mail@lowney.com http://www.lowney.com

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

DRAFT

Supplement No. 1
Soil Property and Interface Direct Shear Test Results
Preliminary Geotechnical Report

Berryessa Creek Levees Project
Santa Clara Valley Water District
Milpitas, California

Report No. 1697-1 has been prepared for:

Winzler & Kelly
495 Tesconi Circle, Santa Rosa, California 95401

January 3, 2002

Robert G. Nixon, P.E. Senior Project Engineer	Fred N. Brovold, P.E., G.E. Senior Project Engineer	C. Barry Butler, P.E., G.E. Principal Geotechnical Engineer Quality Assurance Reviewer

TABLE OF CONTENTS

1.0 INTRODUCTION 1
2.0 SOIL PROPERTIES AND TEST RESULTS 1

TABLE 1 — SUMMARY OF LABORATORY TEST RESULTS

APPENDIX A — LABORATORY TEST RESULTS

SUPPLEMENT NO. 1
SOIL PROPERTY AND INTERFACE DIRECT SHEAR TEST RESULTS
PRELIMINARY GEOTECHNICAL REPORT
BERRYESSA CREEK LEVEES PROJECT
SANTA CLARA VALLEY WATER DISTRICT
MILPITAS, CALIFORNIA

DRAFT

1.0 INTRODUCTION

This supplemental report provides the geogrid interaction coefficient test data discussed in Section 3.1.8 of our Preliminary Geotechnical Report, dated August 28, 2001. The soil property and interface direct shear testing was performed by SGI Testing Services, LLC (SGI), on samples of the existing levee fill obtained during the field investigation. This specialized testing was performed to estimate interaction coefficients between the existing levee fill and geogrid reinforcement material for use in evaluating levee improvement alternatives. The test results are provided in Appendix A.

2.0 SOIL PROPERTIES AND TEST RESULTS

Three bulk samples of the existing levee fill obtained from borings EB-7s, EB-14s and EB-15s were provided to SGI for testing. The samples were selected to represent the range of plasticity exhibited by the levee fill.

Table 1 summarizes the classification, particle size, plasticity and compaction test results for the three samples. Two of the samples were fat clays with PIs of 40 and 42 and one of the samples was a lean clay with sand exhibiting a PI 21. Compaction test (ASTM D 1557) maximum dry densities and optimum moisture contents were 109.5 to 119.5 pounds per cubic foot and 12.5 to 17.0 percent, respectively.

Geogrid interaction coefficients, also named coefficients of direct sliding or pullout resistance factors, are used for analyzing geogrid-reinforced soils. The coefficient of direct sliding is the ratio of the peak or large-displacement shear strength at the test interface between the soil and the geogrid and the peak or large-displacement shear strength of the soil. Interface direct shear testing indicated coefficients of direct sliding at large displacements of 0.42 to 0.77 with an average value of 0.60 and a standard deviation of 0.11. Typical values of coefficients of direct sliding range from about 0.8 to 1.0 for sands and gravels and from about 0.6 to 0.8 for lean clays, silts and clayey sands.

Table 1 – Summary of Test Samples

SGI Sample Number	Boring Location	Classification	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	LL (%)	PL (%)	PI	Maximum Dry Density[1.] (pcf)	Optimum Moisture Content (%)	Coefficients Of Direct Sliding	
												Peak Strength	Large Displacements
1A/B	EB-14s	Fat clay (CH)	54	37	9	0	63	21	42	118.5	12.5	0.53-0.86	0.46-0.70
2A/B	EB-15s	Lean clay with sand (CL)	34	40	26	0	38	17	21	119.5	13.0	0.62-0.85	0.52-0.77
3A/B	EB-7s	Fat clay (CH)	58	36	6	0	68	28	40	109.5	17.0	0.57-0.82	0.51-0.72

Notes:

1. ASTM D 1557

Appendix A

Prepared for:

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services
405 Clyde Avenue
Mountain View, California 94043

FINAL REPORT
SOIL PROPERTY AND INTERFACE DIRECT SHEAR TESTING

BERRYESSA CREEK LEVEE PROJECT

Prepared by:

SGI TESTING SERVICES, LLC

5775 Peachtree Dunwoody Road, Suite 11D
Atlanta, Georgia 30342

Project Number SGI1021

8 October 2001

CAVEAT

The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analysis unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. This testing report is submitted for the exclusive use of the client to whom it is addressed.

1. INTRODUCTION

The details of samples submitted for testing to SGI® Testing Services, LLC (SGI®), 5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342, are as follows:

Submitted by: Mr. Fred Brovold, G.E.

Client: Lowney Associates

Address: 405 Clyde Avenue
Mountain View, California 94043

Materials tested: Geosynthetic Materials: Tensar UX1500HS geogrid
Soil Materials: Sample 1A/B (EB-145), Sample 2A/B (EB-155), and Sample 3A/B (EB-75)

2. TEST PROGRAM

The test procedures and results are described in the following appendices:
Appendix A: Summary of Test Procedures
Appendix B: Soil Test Results
Appendix C: Direct Shear Test Results

3. STORAGE AND DISPOSAL OF MATERIALS

Samples will be stored for 30 days from the date of this report and then discarded unless SGI® is informed otherwise.

* * * * *

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REPORT REVIEW

REPORT PREPARATION BY:	TECHNICAL REVIEW BY:
Robert H. Swan, Jr. President and CEO	Zehong Yuan, Ph.D., P.E. Chief Technical Officer

APPENDIX A

SUMMARY OF TEST PROCEDURES

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SUMMARY OF TEST PROCEDURES

SOIL TESTING

Test Methods

- ***Particle-Size Analysis:*** conducted in accordance with ASTM Standard Test Method D 422, "*Particle-Size Analysis of Soils*", including hydrometer.
- ***Atterberg Limits*:** conducted in accordance with ASTM Standard Test Method D 4318, "*Liquid Limit, Plastic Limit, and Plasticity Index of Soils*".
- ***Soil Classification*:** conducted in accordance with ASTM Standard Test Method D 2487, "Classification of Soils for Engineering Purposes ".
- ***Soil Compaction Testing:*** conducted in accordance with ASTM Standard Test Method D 1557, *"Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lb/ft^3 (2,700 kN-m/m^3))"*.

Test Data Presentation

The soil test results are graphically presented in Appendix B for each of the three soils evaluated.

INTERFACE DIRECT SHEAR TESTING

Test Method

The interface direct shear tests were performed in accordance with the ASTM Standard Test Method D 5321, *"Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method"*. The tests were conducted in a large direct shear device containing an upper and lower shear box. The upper shear box measured 12 in. by 12 in. in plan and 3 in. in depth. The lower shear box measured 12 in. by 14 in. in plan and 3 in. in depth.

Sample Description and Test Configuration

The testing program consisted of three interface direct shear test series. Each interface direct shear test series consisted of three tests, each conducted at a different normal stress. The configuration of the test specimens for each interface direct shear test series is described, from top to bottom, as follows:

- Upper shear box: Sample 1A/B (Test Series 1), Sample 2A/B (Test Series 2), or Sample 3A/B (Test Series 3);
- Tensar UX1500HS geogrid (Test Series 1, 2, and 3); and
- Lower shear box: Sample 1A/B (Test Series 1), Sample 2A/B (Test Series 2), or Sample 3A/B (Test Series 3).

Tested interface: upper soil against geogrid against lower soil

Test Procedures

For each interface direct shear test series, the test specimens were set up in accordance with the above description and tested under the following general test conditions:

- a fresh specimen of lower soil was compacted into the lower box by hand tamping. The initial target compaction conditions (i.e., dry unit weight and moisture content) for each soil material correspond to 90% of maximum dry unit weight and three (3) percentage points above the optimum moisture as determined from the modified Proctor compaction test performed by SGI® and presented in Appendix B.
- a fresh specimen of geogrid was trimmed from the bulk sample and placed on the lower soil but not attached to either of the lower or upper shear boxes.
- a fresh specimen of the upper soil was compacted by hand tamping directly on top of the geogrid specimen. The initial target compaction conditions (i.e., dry unit weight and moisture content) for each soil material correspond to 90% of maximum dry unit weight and three (3) percentage points above the optimum moisture as determined from the modified Proctor compaction test performed by SGI® and presented in Appendix B.
- Test normal stresses: 1000, 2000, and 4000 psf.

- Constant shear displacement rate: 0.04 in/min.
- Shear box size: 12 in. by 12 in.
- each test was conducted under as-placed moisture conditions.
- each geogrid was tested in the machine direction.
- each test was sheared until a minimum total shear displacement of 2 in. was achieved.

Failure Modes

For all three-test series, sliding (shear failure) occurred at the geogrid-lower soil interface during each test.

Test Data Presentation

For each of the interface direct shear tests, the total-stress shearing resistance was evaluated for each applied normal stress. The test data were plotted on a graph of shear force versus horizontal displacement. The resulting plots are presented in Appendix C. The peak value of shear force was used to calculate the peak shear strength. The large displacement shear strength (τ_{LD}) was calculated by using the shear force measured at the end of each test. No area correction was used when computing normal and shear stresses because each test was performed using a constant effective sample area (i.e., the area of the lower shear box was larger than that of the upper shear box).

The calculated shear strengths are summarized in Table 1 of Appendix C and plotted on a graph of shear stress versus normal stress. The results were used to evaluate total-stress peak and large displacement shear strength envelopes. A best-fit straight line was drawn through the data points from each test series to obtain the corresponding total-stress peak and large displacement shear strength friction angles and adhesions. The coefficient of correlation (R^2), a standard statistical indicator of how well the best-fit line matches the test data, was obtained for each best-fit line. The summary plots of shear stress versus normal stress for each test series are also presented in Appendix C. The friction angles, adhesions, and R^2 values derived from the plotted test results are presented in Table 2 of Appendix C.

At the request of Lowney Associates, the coefficients of direct sliding (C_{DS}) at the peak and large-displacement shear strengths were calculated for each interface direct shear test using the following equation:

$$C_{DS} = \frac{\tau_{interface}}{\tau_{soil}}$$

where: $\tau_{interface}$ = peak or large-displacement shear strength of the test interface; and
τ_{soil} = peak or large-displacement shear strength of the soil. The assumed shear strengths of the three soil materials (i.e., soil 1A/B (EB-145), soil 2A/B (EB-155), and soil 3A/B (EB-75) were based on the effective shear strengths measured through consolidated-undrained triaxial compression tests with pore water pressure measurements provided by Lowney Associates. The shear strength of soil 1A/B (EB-145) was estimated from the test data for soil sample EB-3, the shear strength of soil 2A/B (EB-155) was estimated from the test data for soil sample EB-16, and the shear strength of soil 3A/B (EB-145) was estimated from the test data for soil sample EB-7.

The calculated coefficients of direct sliding (C_{DS}) for each interface direct shear test are also summarized in Table 1. The efficiency factors (E_ϕ and E_c) were also calculated for each interface direct shear test series using the following equations:

$$E_\phi = \frac{\tan\delta}{\tan\phi}$$

$$E_c = \frac{a}{c}$$

where: δ = friction angle of the interface test series;
a = adhesion of the interface test series;
ϕ = friction angle of the soil as discussed above; and
c = cohesion of the soil as discussed above.

The calculated efficiency factors for each interface direct shear test series are also summarized in Table 2.

For each test series, it is noted that the reported total-stress shear strength parameters of friction angle and adhesion were determined based on the best-fit straight line drawn through the test data on a plot of shear stress versus normal stress. Caution should be exercised in using these shear strength parameters for applications involving normal stresses outside the range of stresses covered by each test series.

APPENDIX B

SOIL TEST RESULTS

SGI TESTING SERVICES, LLC

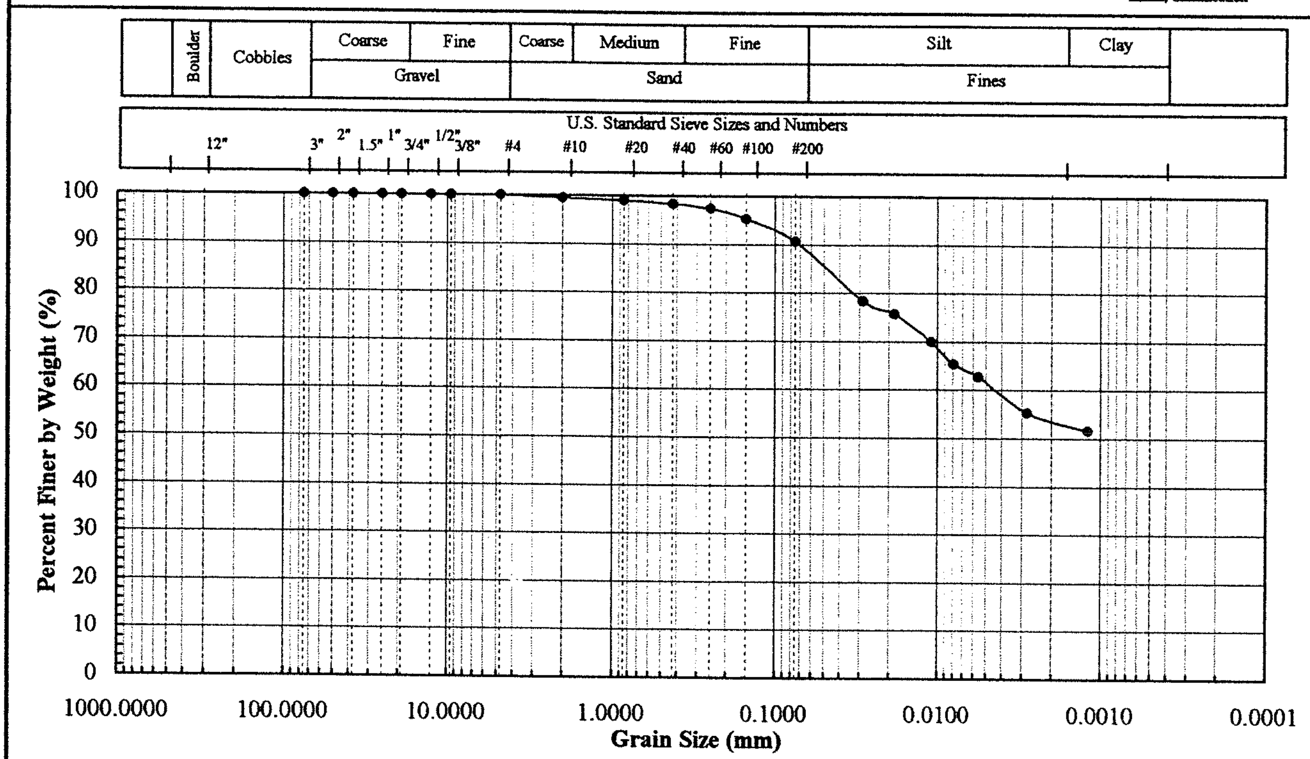
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name: Berryessa Creek Levee
Project No: SGI1021
Client Sample ID: Sample 1A/B (EB-145)
Lab Sample No: AL8517

ASTM D 2216, D 1140, D 422,
C 136, D 4318, D 2487

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

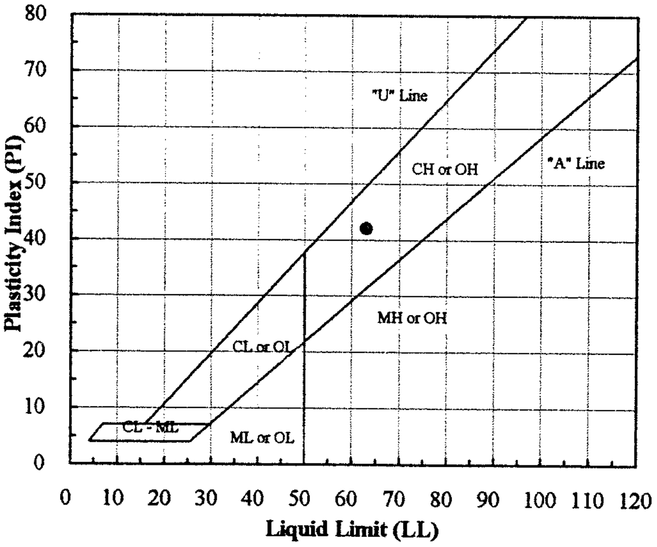


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.5
#20	0.850	98.9
#40	0.425	98.3
#60	0.250	97.3
#100	0.150	95.3
#200	0.075	90.6

Hydrometer Particle Diameter (mm)	% Finer
0.0288	78.3
0.0184	75.6
0.0109	70.0
0.0079	65.4
0.0012	51.6

Gravel (%):	0
Sand (%):	9.4
Fines (%):	90.6
Silt (%):	37.1
Clay (%):	53.5

Coeff. Unif. (Cu):	-
Coeff. Curv. (Cc):	-



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	PL (%)	PI (-)	Engineering Classification
Sample 1A/B (EB-145)	AL8517	23.3	90.6	63	21	42	CH - Fat Clay

Note(s):

al8517.soil.index.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI TESTING SERVICES, LLC

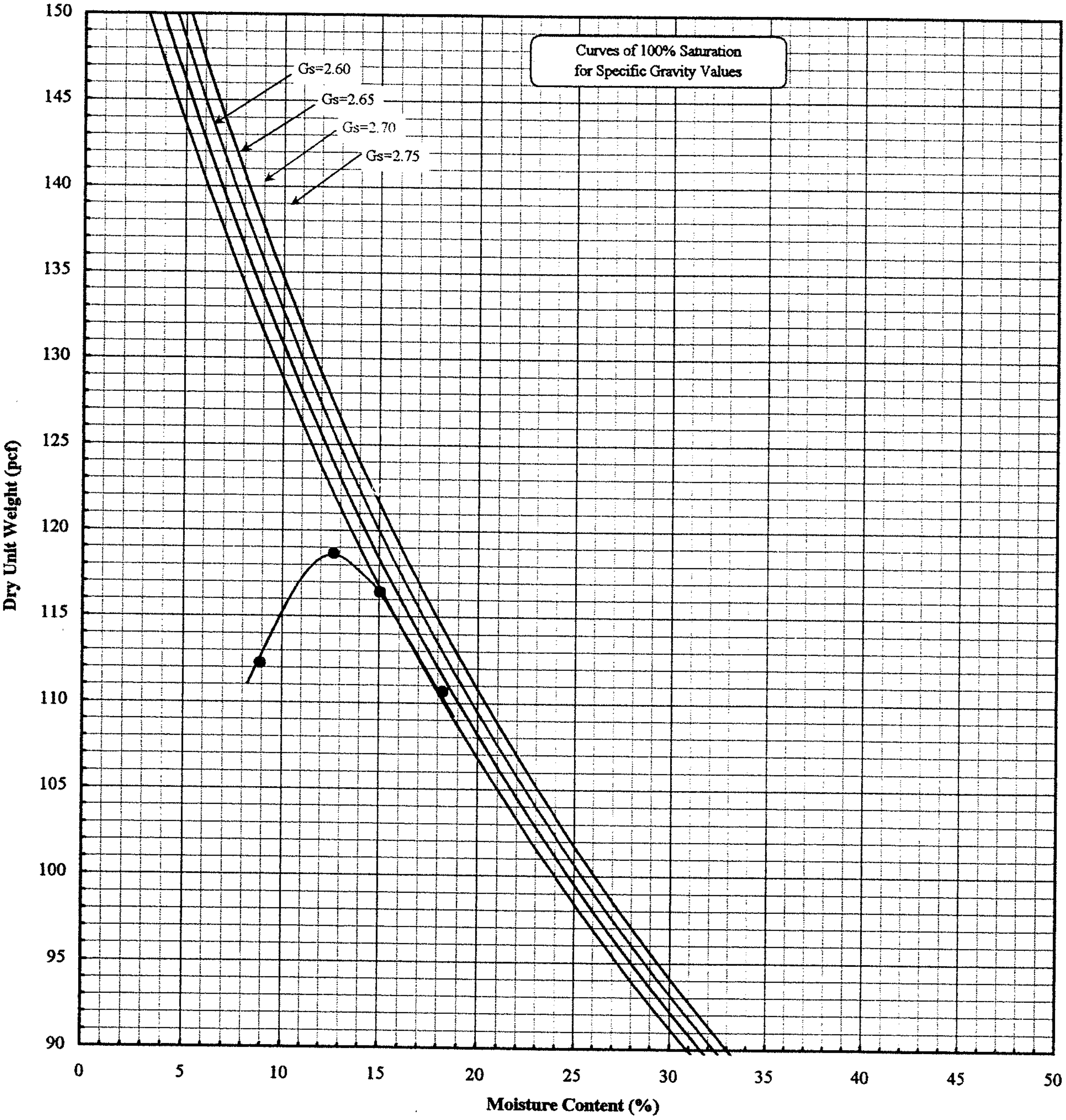
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 1A/B (EB-145)
Lab Sample No:	AL8517

ASTM D1557

COMPACTION MOISTURE-DENSITY RELATIONSHIP

Modified - Method A



Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 1A/B (EB-145)	AL8517	118.5	12.5	

Note(s):

al8517.soil.proctor.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI TESTING SERVICES, LLC

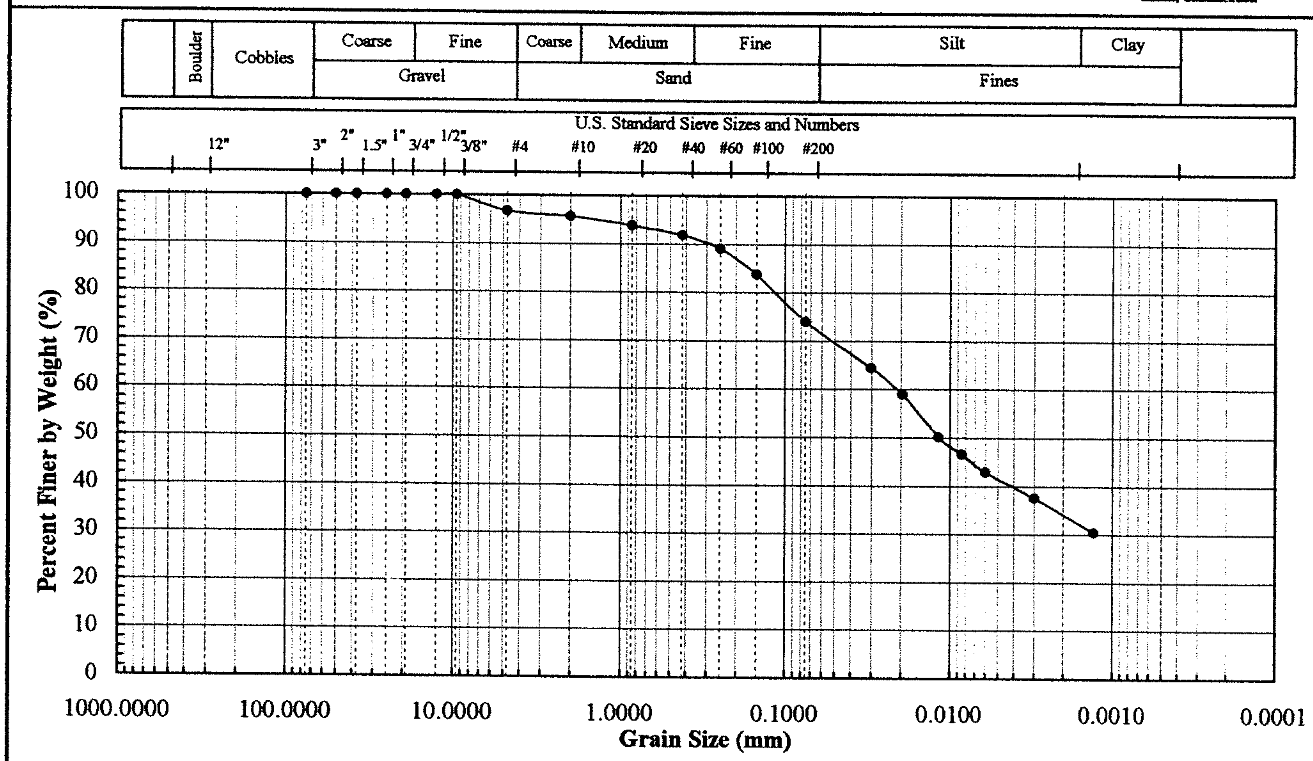
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 2A/B (EB-155)
Lab Sample No:	AL8518

ASTM D 2216, D 1140, D 422,
C 136, D 4318, D 2487

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

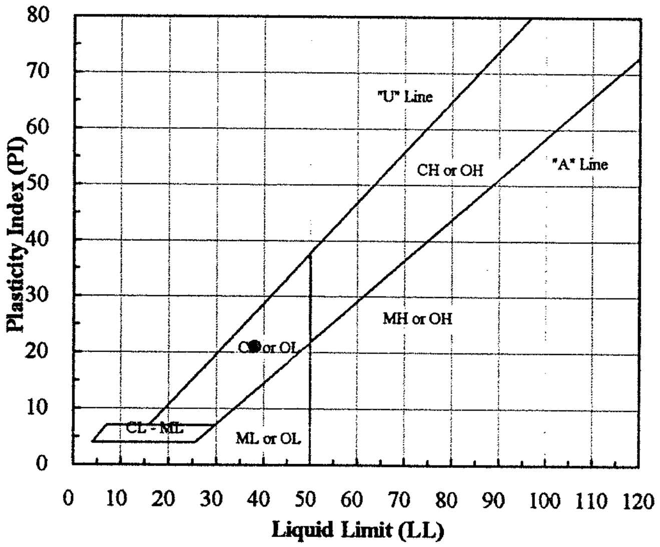


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	96.7
#10	2.00	95.6
#20	0.850	93.8
#40	0.425	91.8
#60	0.250	89.0
#100	0.150	83.7
#200	0.075	73.9

Hydrometer Particle Diameter (mm)	% Finer
0.0303	64.4
0.0196	59.0
0.0117	50.1
0.0084	46.5
0.0013	30.4

Gravel (%):	0
Sand (%):	26.1
Fines (%):	73.9
Silt (%):	39.9
Clay (%):	34.0

Coeff. Unif. (Cu):	-
Coeff. Curv. (Cc):	-



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	Atterberg Limits PL (%)	Atterberg Limits PI (-)	Engineering Classification
Sample 2A/B (EB-155)	AL8518	16.3	73.9	38	17	21	CL - Lean Clay With Sand

Note(s):

al8518.soil.index.xls

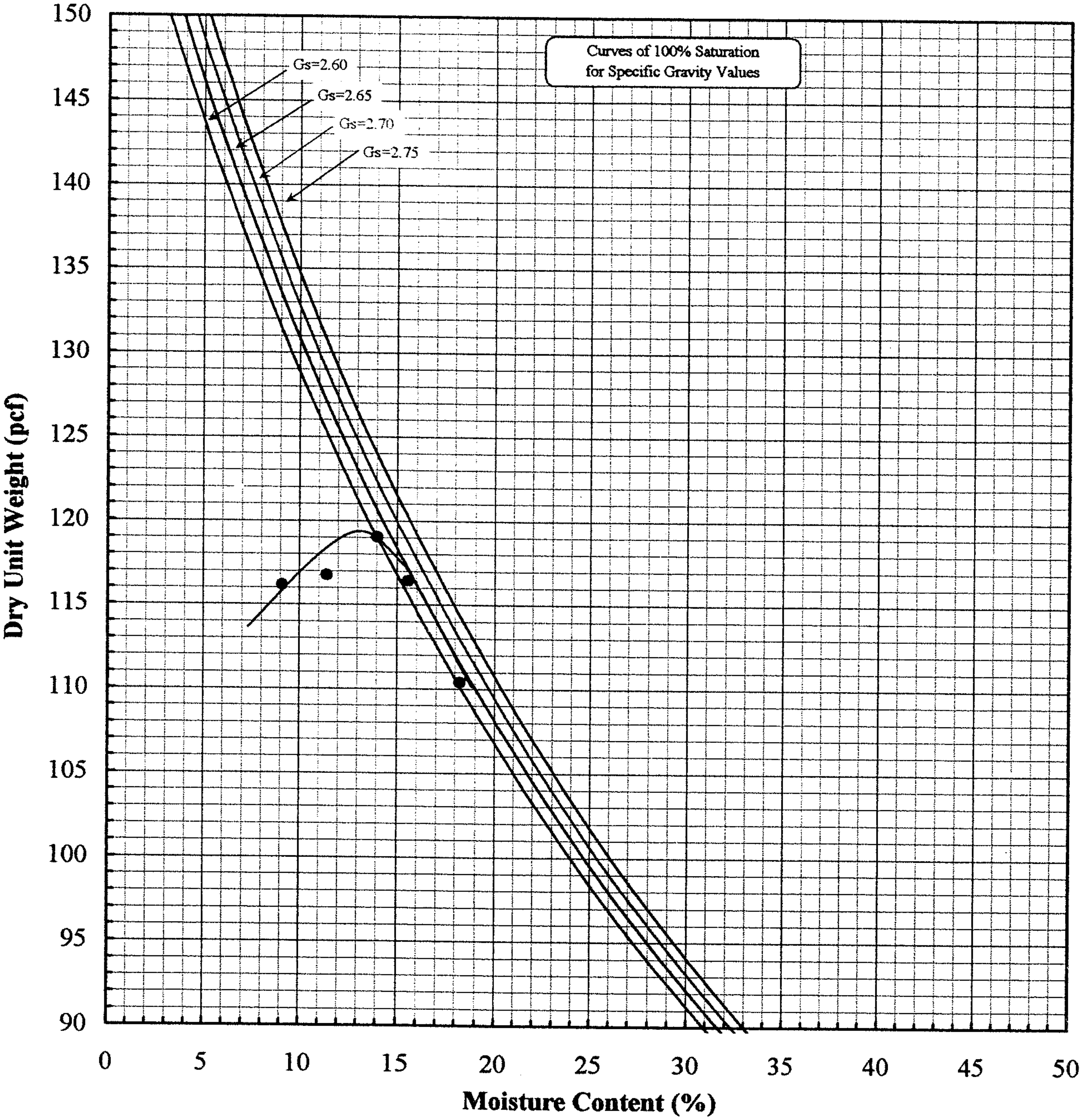
Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI TESTING SERVICES, LLC

5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 2A/B (EB-155)
Lab Sample No:	AL8518

ASTM D1557 **COMPACTION MOISTURE-DENSITY RELATIONSHIP** Modified - Method A



Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 2A/B (EB-155)	AL8518	119.5	13.0	

Note(s):

al8518.soil.proctor.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI TESTING SERVICES, LLC

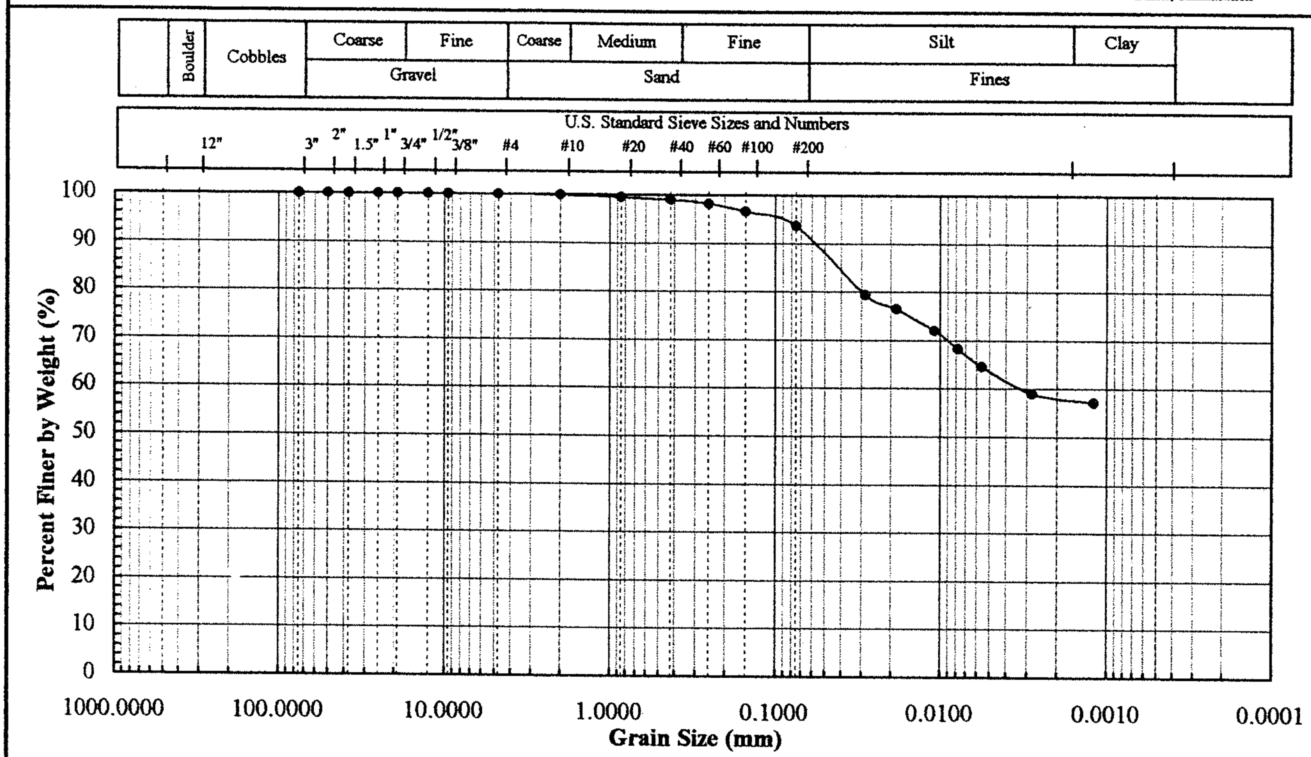
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name:	Berryessa Creek Levee
Project No:	SGI1021
Client Sample ID:	Sample 3A/B (EB-75)
Lab Sample No:	AL8519

ASTM D 2216, D 1140, D 422, C 136, D 4318, D 2487

SOIL INDEX PROPERTIES

Moisture Content, Grain Size, Atterberg Limits, Classification

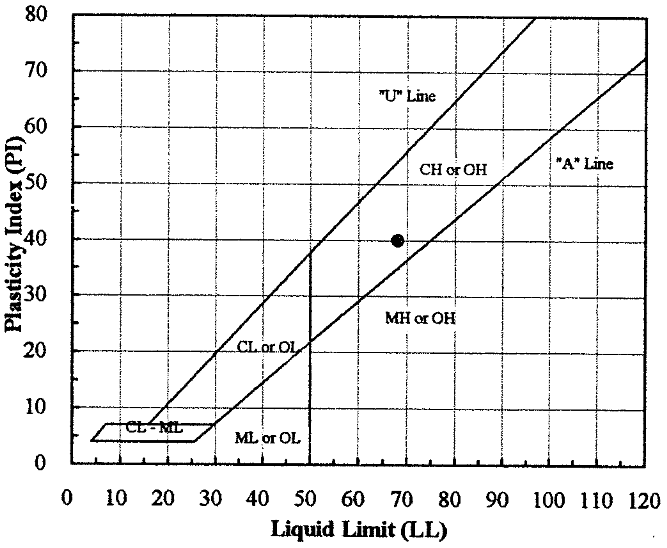


Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
1/2"	12.5	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.5
#40	0.425	98.9
#60	0.250	98.1
#100	0.150	96.6
#200	0.075	93.6

Hydrometer Particle Diameter (mm)	% Finer
0.0285	79.3
0.0184	76.5
0.0108	71.9
0.0078	68.2
0.0012	57.2

Gravel (%):	0
Sand (%):	6.4
Fines (%):	93.6
Silt (%):	35.5
Clay (%):	58.1

Coeff. Unif. (Cu):	-
Coeff. Curv. (Cc):	-



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (%)	PL (%)	PI (-)	Engineering Classification
Sample 3A/B (EB-75)	AL8519	28.6	93.6	68	28	40	CH - Fat Clay

Note(s):

al8519.soil.index.xls

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

SGI TESTING SERVICES, LLC

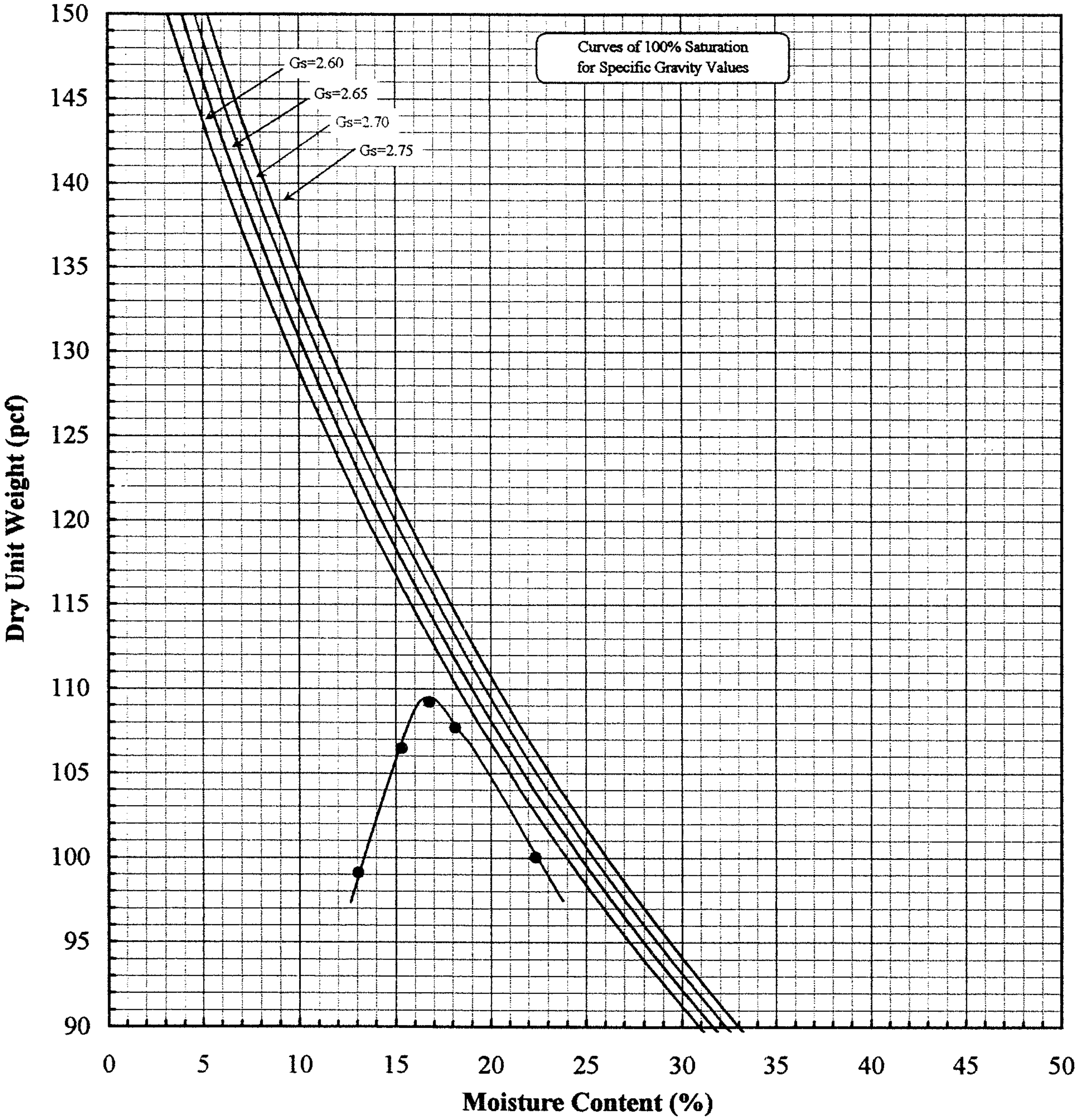
5775 Peachtree Dunwoody Road, Suite 11D, Atlanta, Georgia 30342
Ph: (404) 256 9939 Fax: (404) 705 9300

Project Name: Berryessa Creek Levee
Project No: SGI1021
Client Sample ID: Sample 3A/B (EB-75)
Lab Sample No: AL8519

ASTM D1557

COMPACTION MOISTURE-DENSITY RELATIONSHIP

Modified - Method A



Client Sample ID.	Lab Sample No:	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	Remarks
Sample 3A/B (EB-75)	AL8519	109.5	17.0	

Note(s):

Reviewed by: RHS Date: 8/20/2001
Robert H. Swan, Jr., President and CEO

al8519.soil.proctor.xls

APPENDIX C

INTERFACE DIRECT SHEAR TEST RESULTS

TABLE 1

INTERFACE DIRECT SHEAR TEST RESULTS MEASURED PEAK AND LARGE-DISPLACEMENT TOTAL SHEAR STRENGTHS AND COEFFICIENTS OF DIRECT SLIDING LOWNEY ASSOCIATES – BERRYESSA CREEK LEVEE PROJECT

Test Series Number	Normal[1] Stress (psf)	Measured Peak Shear Strength (psf)	Measured Large-displacement Shear Strength (psf)	Coefficient of Direct Sliding at Peak Strength[2]	Coefficient of Direct Sliding at Large Displacement Strength[2]
1	1000	855	703	0.86	0.70
	2000	1326	1102	0.66	0.55
	4000	2115	1835	0.53	0.46
2	1000	678	618	0.85	0.77
	2000	1026	895	0.69	0.60
	4000	1763	1469	0.62	0.52
3	1000	785	693	0.82	0.72
	2000	1207	1090	0.67	0.61
	4000	1974	1762	0.57	0.51

Notes:
(1) Test specimens were sheared immediately after application of normal stress used for shearing.
(2) The assumed shear strengths of the three soil materials (i.e., soil 1A/B (EB-145), soil 2A/B (EB-155), and soil 3A/B (EB-75) were based on the effective shear strengths measured through consolidated-undrained triaxial compression tests with pore water pressure measurements provided by Lowney Associates. The shear strength of soil 1A/B (EB-145) was estimated from the test data for soil sample EB-3, the shear strength of soil 2A/B (EB-155) was estimated from the test data for soil sample EB-16, and the shear strength of soil 3A/B (EB-145) was estimated from the test data for soil sample EB-7.

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TABLE 2
INTERFACE DIRECT SHEAR TEST RESULTS
MEASURED TOTAL STRESS STRENGTH PARAMETERS
LOWNEY ASSOCIATES – BERRYESSA CREEK LEVEE PROJECT

Test Series Number	Interfaces Tested [(1)]	Normal Stress (psf)	Peak Strength[(2)]				Large Displacement Shear Strength[(3)]				Reference Appendix Figure Numbers
			Friction Angle	Adhesion/ Cohesion (psf)	R^2	$E\phi, E_c$	Friction Angle	Adhesion/ Cohesion (psf)	R^2	$E\phi, E_c$	
1	Soil sample 1A/B (EB-145) against Tensar UX1500HS geogrid against Soil sample 1A/B (EB-145) Under As-Placed Moisture Conditions	1000 to 4000	23°	460	0.998	0.42,460	21°	335	1.000	0.38, 335	C-1
2	Soil sample 2A/B (EB-155) against Tensar UX1500HS geogrid against Soil sample 2A/B (EB-155) Under As-Placed Moisture Conditions	1000 to 4000	20°	310	0.998	0.53, 2.58	16°	330	1.000	0.42, 2.75	C-2
3	Soil sample 3A/B (EB-75) against Tensar UX1500HS geogrid against Soil sample 3A/B (EB-75) Under As-Placed Moisture Conditions	1000 to 4000	22°	400	0.999	0.48, 3.33	19°	355	0.998	0.41, 2.96	C-3

Note:

(1) See Appendix A for detailed test conditions and procedures.

(2) The reported total-stress shear strength parameters for each test series were determined from a best-fit line drawn through the test data. Caution should be exercised in using these shear strength parameters for applications involving normal stresses outside the range of stresses covered by the test series. The value of R^2, the coefficient of correlation, provides an indication of how well the best-fit shear strength parameters match the test data. The assumed shear strengths of the three soil materials (i.e., soil 1A/B (EB-145), soil 2A/B (EB-155), and soil 3A/B (EB-75) were based on the effective shear strengths measured through consolidated-undrained triaxial compression tests with pore water pressure measurements provided by Lowney Associates. The shear strength of soil 1A/B (EB-145) was estimated from the test data for soil sample EB-3, the shear strength of soil 2A/B (EB-155) was estimated from the test data for soil sample EB-16, and the shear strength of soil 3A/B (EB-145) was estimated from the test data for soil sample EB-7.

(3) The large displacement shear strength (τ_{LD}) was calculated using the shear force measured at the end of each test.

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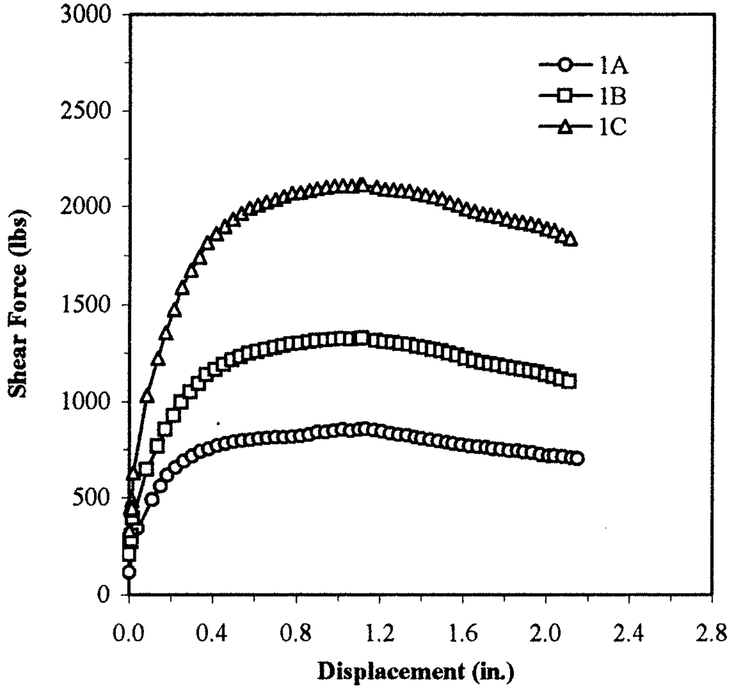
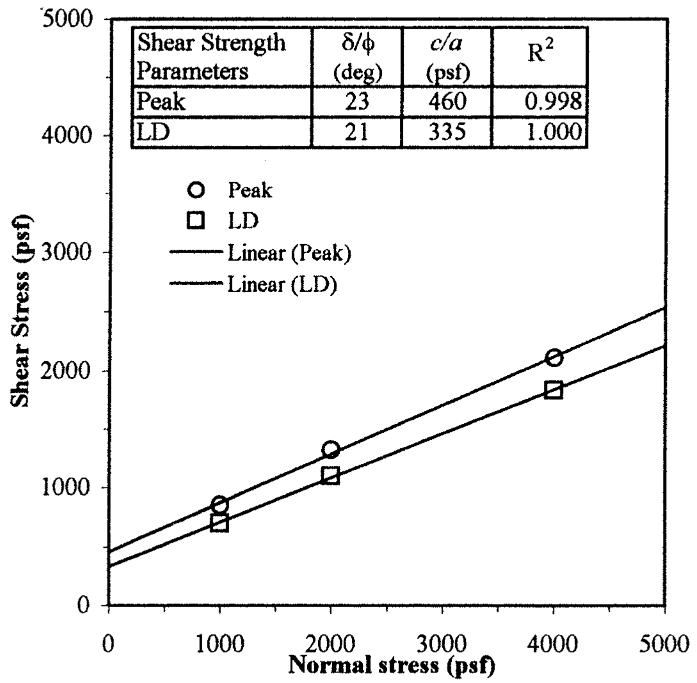
LOWNEY ASSOCIATES - BERRYESSA CREEK LEVEE PROJECT
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 1: soil sample 1A/B (EB-145) against Tensar UX1500HS geogrid against soil sample 1A/B (EB-145) under as-placed moisture conditions





Shear Strength Parameters	δ/ϕ (deg)	*c/a* (psf)	R^2
Peak	23	460	0.998
LD	21	335	1.000

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{dl} (pcf)	ω_l (%)	ω_f (%)	γ_{dl} (pcf)	ω_l (%)	ω_f (%)	ω_l (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	
1A	12 x 12	1000	0.040	-	-	-	-	106.7	15.5	15.3	106.7	15.5	15.3	-	-	855	703	(1)
1B	12 x 12	2000	0.040	-	-	-	-	106.5	15.7	15.4	106.5	15.7	15.4	-	-	1326	1102	(1)
1C	12 x 12	4000	0.040	-	-	-	-	106.6	15.6	15.2	106.6	15.6	15.2	-	-	2115	1835	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the geogrid and the lower soil interface during each test.

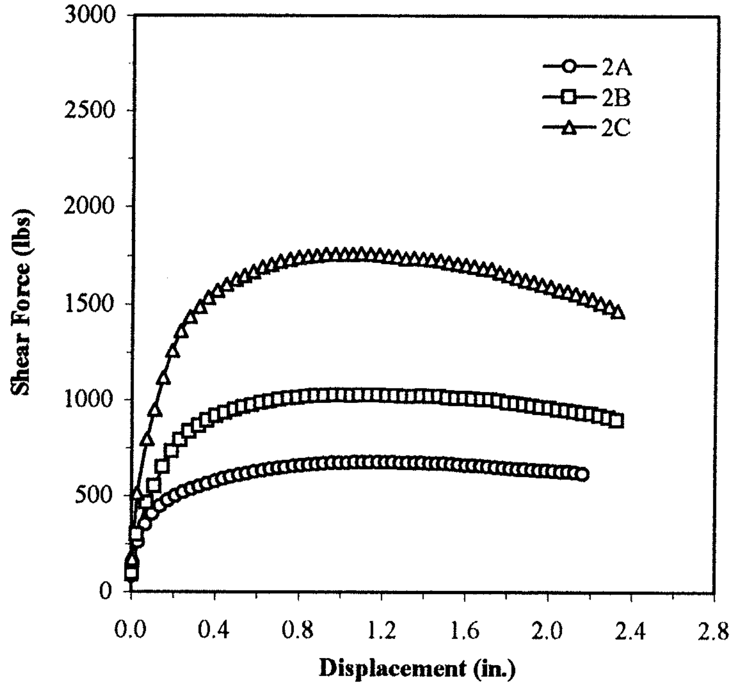
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

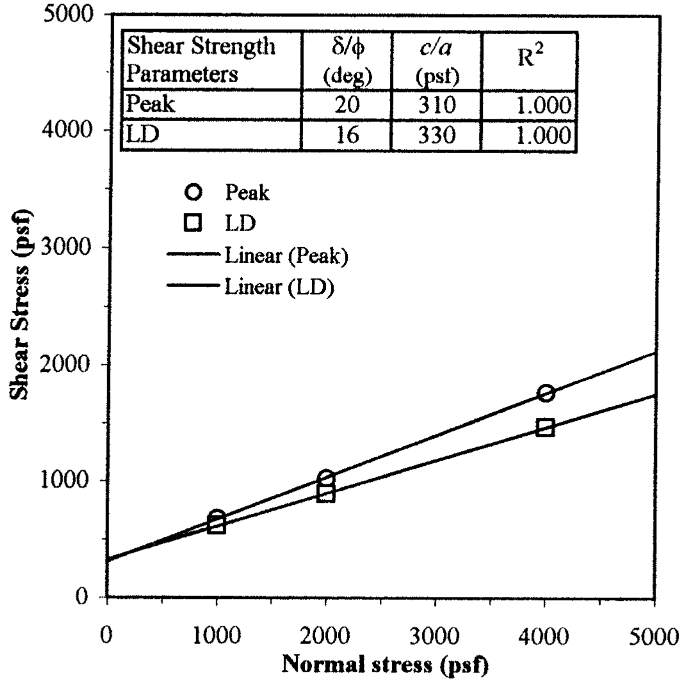
SGI TESTING SERVICES, LLC

DATE OF TEST:	24 to 25 August 2001
FIGURE NO.	C-1
PROJECT NO.	SGI1021
DOCUMENT NO.	SGI01111
FILE NO.	

LOWNEY ASSOCIATES - BERRYESSA CREEK LEVEE PROJECT
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 2: soil sample 2A/B (EB-155) against Tensar UX1500HS geogrid against soil sample 2A/B (EB-155) under as-placed moisture conditions





Shear Strength Parameters	δ/ϕ (deg)	*c/a* (psf)	R^2
Peak	20	310	1.000
LD	16	330	1.000

Test	Shear	Normal	Shear	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure
No.	Box Size (in. x in.)	Stress (psf)	Rate (in./min)	Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{dl} (pcf)	ω_l (%)	ω_f (%)	γ_{dl} (pcf)	ω_l (%)	ω_f (%)	ω_l (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	Mode
2A	12 x 12	1000	0.040	-	-	-	-	107.2	16.4	16.3	107.2	16.4	16.3	-	-	678	618	(1)
2B	12 x 12	2000	0.040	-	-	-	-	107.7	15.9	15.8	107.7	15.9	15.8	-	-	1026	895	(1)
2C	12 x 12	4000	0.040	-	-	-	-	107.4	16.2	16.0	107.4	16.2	16.0	-	-	1763	1469	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the geogrid and the lower soil interface during each test.

(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

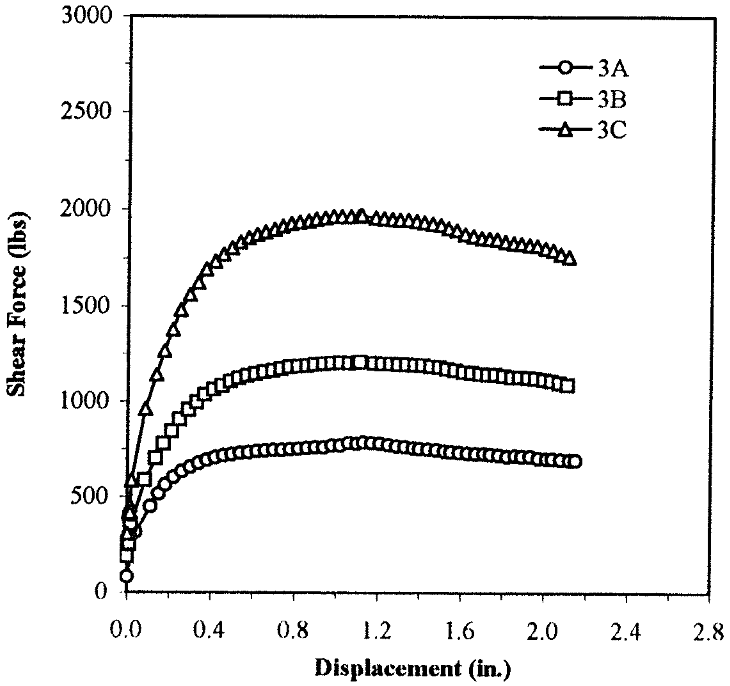
DATE OF TEST: 5 September 2001

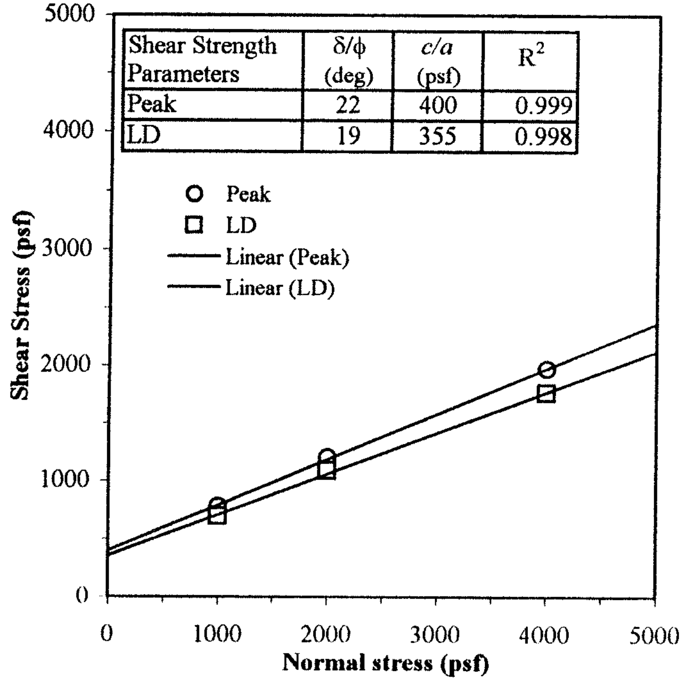
SGI TESTING SERVICES, LLC

FIGURE NO.	C-2
PROJECT NO.	SGI1021
DOCUMENT NO.	SGI01111
FILE NO.	

LOWNEY ASSOCIATES - BERRYESSA CREEK LEVEE PROJECT
INTERFACE DIRECT SHEAR TESTING (ASTM D 5321)

Test Series 3: soil sample 3A/B (EB-75) against Tensar UX1500HS geogrid against soil sample 3A/B (EB-75) under as-placed moisture conditions





Shear Strength Parameters	δ/ϕ (deg)	*c/a* (psf)	R^2
Peak	22	400	0.999
LD	19	355	0.998

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_{dl} (pcf)	ω_i (%)	ω_f (%)	γ_{dl} (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_p (psf)	τ_{LD} (psf)	
3A	12 x 12	1000	0.040	-	-	-	-	98.1	20.6	20.6	98.1	20.6	20.6	-	-	785	693	(1)
3B	12 x 12	2000	0.040	-	-	-	-	98.4	20.3	20.3	98.4	20.3	20.3	-	-	1207	1090	(1)
3C	12 x 12	4000	0.040	-	-	-	-	98.3	20.4	20.4	98.3	20.4	20.4	-	-	1974	1762	(1)

Notes: (1) Sliding (i.e., shear failure) occurred between the geogrid and the lower soil interface during each test.
(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

SGI TESTING SERVICES, LLC

DATE OF TEST:	10 to 11 September 2001
FIGURE NO.	C-3
PROJECT NO.	SGI1021
DOCUMENT NO.	SGI01111
FILE NO.	